Tahmoor Colliery
Longwalls 25 to 26

WOLLONDILLY SHIRE COUNCIL ROADS, BRIDGES AND CULVERTS

SURFACE SAFETY AND SERVICEABILITY MANAGEMENT PLAN

REVISION C

Mine Subsidence Engineering Consultants
Level 1 228 Victoria Avenue – Chatswood – NSW 2067
PO Box 3047 – Willoughby North – NSW 2068
Tel. (02) 9413 3777 Fax. (02) 9413 3822
Email: enquiries@minesubsidence.com

www.minesubsidence.com

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GENERAL

AUTHORISATION OF SURFACE SAFETY AND SERVICEABILITY MANAGEMENT PLAN

Authorised on behalf of Tahmoor Colliery:	
Name:	IAN SHEPPARD
Signature:	
Position:	MANAGER ENVIRONMENT & COMMUNITY
Date:	28-11-08.

Authorised on behalf of Wollondilly Shire Council:	
Name:	Justin Nyholm
Signature:	
Position:	Manager Works
Date:	28-11-08

REVIEW

Date	Rev	Comments
Mar-06	A	Draft for Submission to Wollondilly Shire Council
Aug-06	B	Chapter 1 amended and info re Thirlmere Way Overbridge updated, as agreed in Plan Review Meeting, 7 August 2006
Nov-08	C	Updated for Castlereagh Street Bridge

REFERENCES

AS/NZS 4360:1999 Risk Management.

MSEC, (2006) *Tahmoor Colliery Longwalls 24 to 26 - The Prediction of Subsidence Parameters and the Assessment of Mine Subsidence Impacts on Surface and Sub-Surface Features due to mining Longwalls 24 To 26 at Tahmoor Colliery in support of an SMP Application.* (Report MSEC157), prepared by Mine Subsidence Engineering Consultants, 2006.

JMA, (2006) Structural Investigation Report: for the Castlereagh Street Bridge over Myrtle Creek, Tahmoor subject to Prescribed Ground Movements Caused By Predicted Mines Subsidence. Report No. JM061109-Rev 1, John Matheson & Associates, 2006.

JMA, (2007) Strengthening Measures for the Abutment Walls, to Castlereagh Street Bridge over Myrtle Creek, Tahmoor subject to Prescribed Ground Movements Caused By Predicted Mines Subsidence. Report No. JM070810, John Matheson & Associates, 2007.

JMA, (2008) *Castlereagh Street Bridge, Tahmoor: Trigger Report.* Report No. JM081112 Rev B. John Matheson & Associates, 2008. (attached in Appendix E of this Plan)

TABLE OF CONTENTS

Drawings

Drawings referred to in this report are included in Appendix B at the end of the report.

Drawing No.	*Description*	*Rev*
MSEC286-040201	Local Roads	C
MSEC286-040202	Bridges, Tunnels and Culverts	C

CHAPTER 1. INTRODUCTION

1.1. Background

Tahmoor Colliery is located approximately 80 kilometres south west of Sydney in the township of Tahmoor NSW. It is managed and operated by Xstrata Coal. Tahmoor Colliery has previously mined 24 longwalls to the north and west of the mine's current location.

Longwalls 25 to 26 are a continuation of a series of longwalls that extend into the Tahmoor North Lease area, which began with Longwall 22. The longwall panels are located between the Bargo River in the south-east, the township of Thirlmere in the west and Picton in the north. A portion of each longwall is located beneath the urban area of Tahmoor. Infrastructure owned by Wollondilly Shire Council is located within these areas.

This Management Plan provides detailed information about how the risks associated with the mining beneath sewer infrastructure will be managed by Tahmoor Colliery and Wollondilly Shire Council.

The Management Plan is a live document that can be amended at any stage of mining, to meet the changing needs of Tahmoor Colliery and Wollondilly Shire Council

1.2. Predicted Subsidence Movements

A summary of the predicted maximum incremental parameters over the whole subsided area due to the extraction of each longwall, is shown in Table 1.1.

Table 1.1 Maximum Predicted Incremental Subsidence Parameters

Subsidence Parameter	LW 22	LW 23	LW 24	LW 25	LW 26
Vertical Subsidence (mm)	503	613	596	631	636
Transverse Tilt (mm/m)	3.5	4.9	4.7	5.0	5.1
Longitudinal Tilt (mm/m)	3.0	3.8	3.5	3.7	3.7
Transverse Tensile Strain (mm/m)	0.4	0.7	0.7	0.8	0.8
Longitudinal Tensile Strain (mm/m)	0.6	0.7	0.8	0.8	0.8
Transverse Compressive Strain (mm/m)	0.9	1.6	1.5	1.7	1.7
Longitudinal Compressive Strain (mm/m)	0.6	0.8	0.6	0.6	0.8
Transverse Hogging Curvature (km^{-1})	0.03	0.05	0.05	0.05	0.05
Longitudinal Hogging Curvature (km^{-1})	0.04	0.05	0.05	0.05	0.05
Transverse Sagging Curvature (km^{-1})	0.06	0.11	0.10	0.11	0.11
Longitudinal Sagging Curvature (km^{-1})	0.04	0.05	0.04	0.04	0.05

The maximum predicted cumulative subsidence parameters, after the extraction of each longwall, are shown in **Error! Reference source not found.**.

Table 1.2 Maximum Predicted Cumulative Subsidence Parameters

Subsidence Parameter	LW 22	LW 23	LW 24	LW 25	LW 26
Vertical Subsidence (mm)	503	756	850	892	934
Transverse Tilt (mm/m)	3.5	5.0	4.8	5.2	5.2
Longitudinal Tilt (mm/m)	3.0	4.4	4.9	5.1	5.2
Transverse Tensile Strain (mm/m)	0.4	0.7	0.7	1.0	1.3
Longitudinal Tensile Strain (mm/m)	0.6	0.7	0.8	0.9	0.9
Transverse Compressive Strain (mm/m)	0.9	1.6	1.7	1.7	1.8
Longitudinal Compressive Strain (mm/m)	0.6	0.8	0.8	0.8	0.8
Transverse Hogging Curvature (km^{-1})	0.03	0.05	0.05	0.07	0.09
Longitudinal Hogging Curvature (km^{-1})	0.04	0.05	0.05	0.06	0.06
Transverse Sagging Curvature (km^{-1})	0.06	0.11	0.11	0.11	0.12
Longitudinal Sagging Curvature (km^{-1})	0.04	0.05	0.05	0.05	0.05

1.3. Limitations

This Management Plan is based on the predictions of the effects of mining on surface infrastructure as provided in Report No. MSEC157 by Mine Subsidence Engineering Consultants. Predictions are based on the planned configuration of longwalls at Tahmoor Colliery (as shown in Drawing No. MSEC286-040201), along with available geological information and data from numerous subsidence studies for longwalls previously mined in the area.

Infrastructure considered in this Plan has been identified from aerial photographs, regional maps, design drawings and from discussions between Centennial representatives and Wollondilly Shire Council personnel.

The impacts of mining on surface and sub-surface features have been assessed in detail. However, it is recognised that the prediction and assessment of subsidence can be relied upon only to a certain extent. The limitations of the prediction and assessment of mine subsidence are discussed in report MSEC157 by Mine Subsidence Engineering Consultants.

As discussed in the report, there is a low probability that ground movements and their impacts could exceed the predictions and assessments. However, if these potentially higher impacts are considered prior to mining, they can be managed.

1.4. Objectives

The objectives of this Surface Safety and Serviceability Management Plan (SSSMP) are to establish procedures to measure, control, mitigate and repair potential impacts that might occur to roads, bridges and culverts.

The objectives of the SSSMP have been developed to:-

- Ensure the safe and serviceable operation of all surface infrastructure. Public and workplace safety is paramount. Disruption and inconvenience should be kept to minimal levels.
- Monitor ground movements and the condition of surface infrastructure during mining.
- Initiate action to mitigate or remedy potential significant impacts that are expected to occur on the surface.
- Provide a plan of action in the event that the impacts of mine subsidence are greater than those that are predicted.
- Provide a forum to report, discuss and record impacts to the surface. This will involve Tahmoor Colliery, Council, Mine Subsidence Board, Department of Mineral Resources, and consultants as required.
- Establish lines of communication and emergency contacts.

1.5. Scope

The SSSMP is to be used to protect and monitor the condition of the items of infrastructure identified to be at risk due to mine subsidence. The major items at risk are:-

- Local roads
- Bridges
- Culverts

The SSSMP only covers infrastructure that is located within the general application area, which defines the extent of land that may be affected by mine subsidence as a result of mining Longwalls 25 to 26. The management plan does not include other roads, bridges and culverts owned by Wollondilly Shire Council which lie outside the extent of the general application area.

1.6. Proposed Mining Schedule

It is planned that each longwall will extract coal working northwest from the southeastern ends. This SSSMP covers longwall mining until completion of mining in Longwall 26 and for sufficient time thereafter to allow for completion of subsidence effects.

The current schedule of mining is shown in Table 1.3.

Table 1.3 Schedule of Mining

Longwall	Start Date	Completion Date
Longwall 25	August 2008	August 2009
Longwall 26	October 2009	October 2010

1.7. Definition of Active Subsidence Zone

As a longwall progresses, subsidence begins to develop at a point in front of the longwall face and continues to develop after the longwall passes. The majority of subsidence movement typically occurs within an area 150 metres in front of the longwall face to an area 450 metres behind the longwall face.

This is termed the "active subsidence zone" for the purposes of this SSSMP, where surface monitoring is generally conducted. The active subsidence zone for each longwall is defined by the area bounded by the predicted 20 mm subsidence contour for the active longwall and a distance of 150 metres in front and 450 metres behind the active longwall face, as shown by Fig. 1.1.

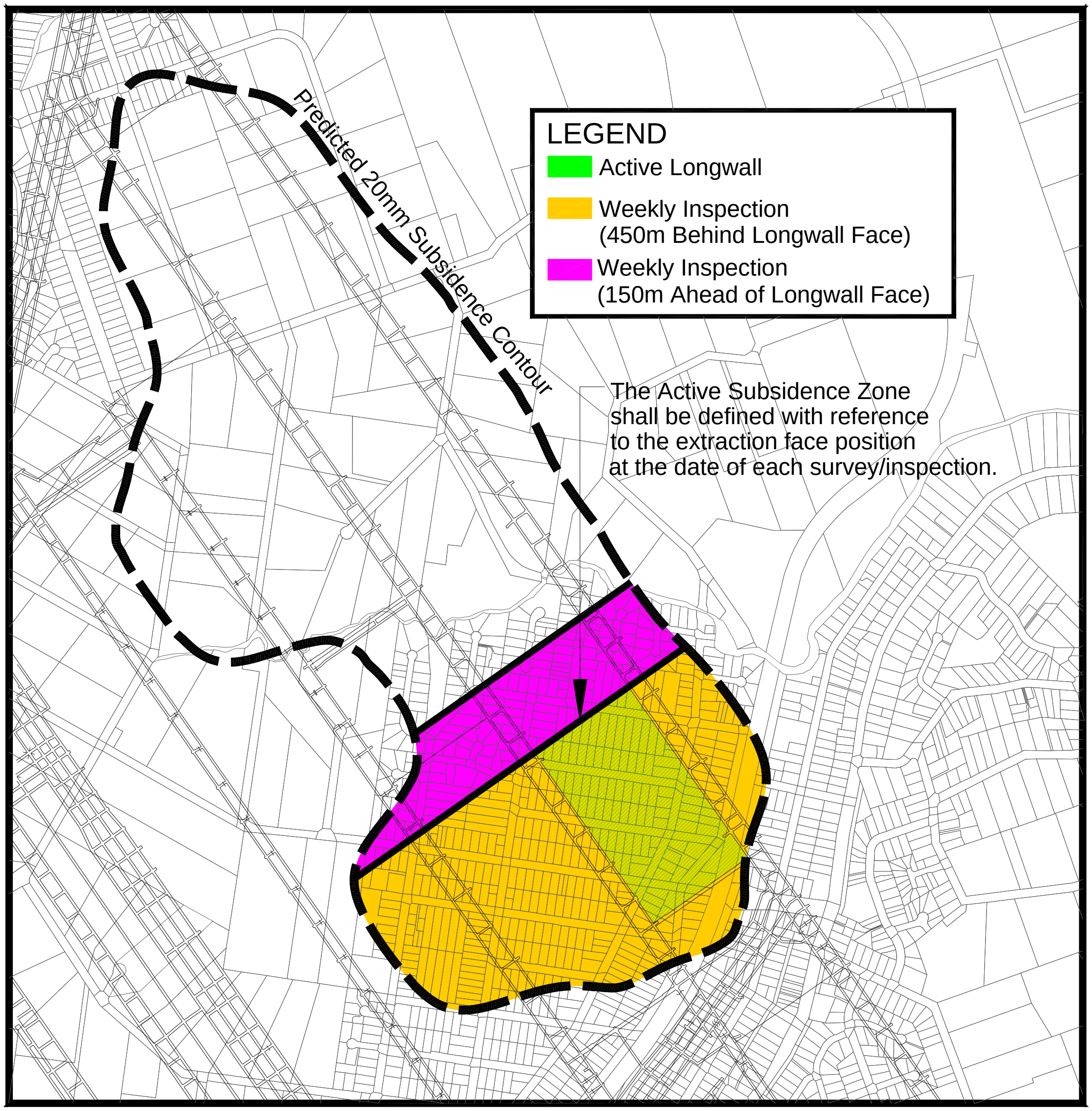

Fig. 1.1 Diagrammatic Representation of Active Subsidence Zone

CHAPTER 2. RISK MANAGEMENT METHOD

2.1. General

The Australian/New Zealand standard for Risk Management defines the terms used in the risk management process, which includes the identification, analysis, assessment, treatment and monitoring of risk. In this context:-

2.1.1. Consequence

'The outcome of an event expressed qualitatively or quantitatively, being a loss, injury, disadvantage or gain. There may be a range of possible outcomes associated with an event.'[1] The consequences of a hazard are rated from very slight to very severe.

2.1.2. Likelihood

'Used as a qualitative description of probability or frequency.'[2] The likelihood can range from very rare to almost certain.

2.1.3. Hazard

'A source of potential harm or a situation with a potential to cause loss.'[3]

2.1.4. Risk

'The chance of something happening that will have an impact upon objectives. It is measured in terms of consequences and likelihood.'[4] The risk combines the likelihood of an impact occurring with the consequence of the impact occurring. The risk is rated from very low to extreme. In this study, the likelihood and consequence are combined via the qualitative risk analysis matrix shown in Table 2.1, to determine an estimated level of risk for particular events or situations.

The Risk Analysis Matrix is similar to the example provided in AS/NZS 4360:1995, Appendix D, p.25.

Table 2.1 Qualitative Risk Analysis Matrix

LIKELIHOOD	CONSEQUENCES				
	Very Slight	**Slight**	**Moderate**	**Severe**	**Very Severe**
Almost Certain	Low	Moderate	High	Extreme	Extreme
Likely	Low	Moderate	High	Very High	Extreme
Moderate	Low	Low	Moderate	High	Very High
Unlikely	Very Low	Low	Moderate	High	High
Rare	Very Low	Very Low	Low	Moderate	High
Very Rare	Very Low	Very Low	Low	Moderate	Moderate

This SSSMP adopts a common system of nomenclature to summarise each risk analysis, which is **"LIKELIHOOD / CONSEQUENCE → LEVEL OF RISK".**

For example, if the likelihood of a risk is assessed as "**UNLIKELY**", and the consequence of a risk is assessed as "**SEVERE**", the risk analysis would be summarised as **"UNLIKELY / SEVERE → HIGH"**.

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[1] AS/NZS 4360:1999 – Risk Management pp2
[2] AS/NZS 4360:1999 – Risk Management pp2
[3] AS/NZS 4360:1999 – Risk Management pp2
[4] AS/NZS 4360:1999 – Risk Management pp3

CHAPTER 3. RISK ASSESSMENT

3.1. Local Roads

There are a number of local roads within the SMP Area, as shown in Drawing No. MSEC286-040201.

The main road is Remembrance Drive (formerly the Hume Highway), which connects Tahmoor with Picton to the north, and Bargo to the south. Some main services infrastructure is located along Remembrance Drive, and includes gas mains, water mains, and optical fibre cables. The main retail and commercial buildings are also located along Remembrance Drive. Remembrance Drive crosses over Longwalls 24 to 26.

The other significant road within the SMP Area is Thirlmere Way, which connects Thirlmere with Tahmoor to the east, and Picton to the northeast. A small section of Thirlmere Way crosses over Longwall 25, and it has already been undermined by Longwalls 22, 23A and 24B.

The network of local roads is spread across the entire SMP Area, and therefore, will experience the full range of subsidence impacts.

Predictions of systematic subsidence, tilt and strain were made along two major roads, Remembrance Drive and Thirlmere Way, which are shown in Fig. 3.1 and Fig. 3.2, respectively. A summary of the maximum predicted systematic subsidence, tilt and strain along these roads, due to the extraction of Longwalls LW22 to LW26, is provided in Table 3.1. This table also provides the maximum predicted subsidence, tilt and strain within the entire SMP Area, which can be used for conservative predictions of maximum ground movements across the remaining road network.

Table 3.1 Maximum Predicted Systematic Subsidence, Tilt and Strain along the Major Roads due to the Extraction of Longwalls LW22 to LW26

Location	Maximum Predicted Subsidence (mm)	Maximum Predicted Tilt (mm/m)	Maximum Predicted Tensile Strain (mm/m)	Maximum Predicted Compressive Strain (mm/m)
Entire SMP Area	934	5.2	1.3	1.8
Remembrance Drive	768	4.1	0.5	0.8
Thirlmere Way	913	3.4	0.8	0.9

The maximum predicted tilt of 5.2 mm/m, or a change in gradient of 0.5% is very small considering that sealed roads are usually constructed with gradients of approximately 3.0%. The resulting change in road superelevation or gradient is unlikely to affect the serviceability of the road.

The maximum predicted tensile and compressive strains are 1.3 mm/m and 1.8 mm/m respectively. It is possible that hairline cracks may occur in some places due to concentration of tensile strains and that minor localised buckling of the road surface may occur in other places, due to concentration of compressive strains. However, because the predicted strains are relatively low, any such problems are likely to be infrequent occurrences and minor in nature.

Monitoring of road pavements has been undertaken at Tahmoor during the extraction of Longwalls 22, 23A, 23B, 24A and 24B at Tahmoor Colliery. The monitoring includes a network of ground monitoring lines and weekly visual inspections in areas that are experiencing active subsidence. Approximately 10.3 kilometres of asphaltic pavement lie directly above the extracted longwalls and a total of 12 impacts have been observed. One of these impact sites, located on Lintina Street, was substantially greater than the other 11 impact sites. The observed rate of impact equates to an average of one impact for every 1000 metres of pavement.

There have also been impacts to concrete kerbs, gutters and footpaths at Tahmoor. The frequency of impacts is higher than asphaltic pavements, which is understandable as these concrete structures are more vulnerable to mine subsidence movements as they are long, thin and brittle structures.

Traffic signs and other road infrastructure should not suffer any damage due to mine subsidence.

3.1.1. Risk Assessment

The risk to local sealed roads is that deformation (cracking, buckling or wrinkling) of the road surface may occur. Four levels of impact, in increasing order of severity, have been identified for risk analysis.

1. Minor deformations (cracks less than 2 mm), occurring infrequently within the road network
2. Minor deformations (cracks less than 2 mm), occurring extensively within the road network
3. Major deformations (cracks greater than 2 mm), occurring infrequently within the road network
4. Major deformations (cracks greater than 2 mm), occurring extensively within the road network

Table 3.2 summarises the risk analysis for local sealed roads.

Table 3.2 Risk Analysis for Local Sealed Roads

Level of Impact	Likelihood	Consequence	Level of Risk
Infrequent, minor deformations	LIKELY	SLIGHT	MODERATE
Frequent, minor deformations	UNLIKELY	MODERATE	MODERATE
Infrequent, major deformations	UNLIKELY	MODERATE	MODERATE
Frequent, major deformations	VERY RARE	SEVERE	MODERATE

Any damage to local roads will be repaired at the expense of the Mine Subsidence Board.

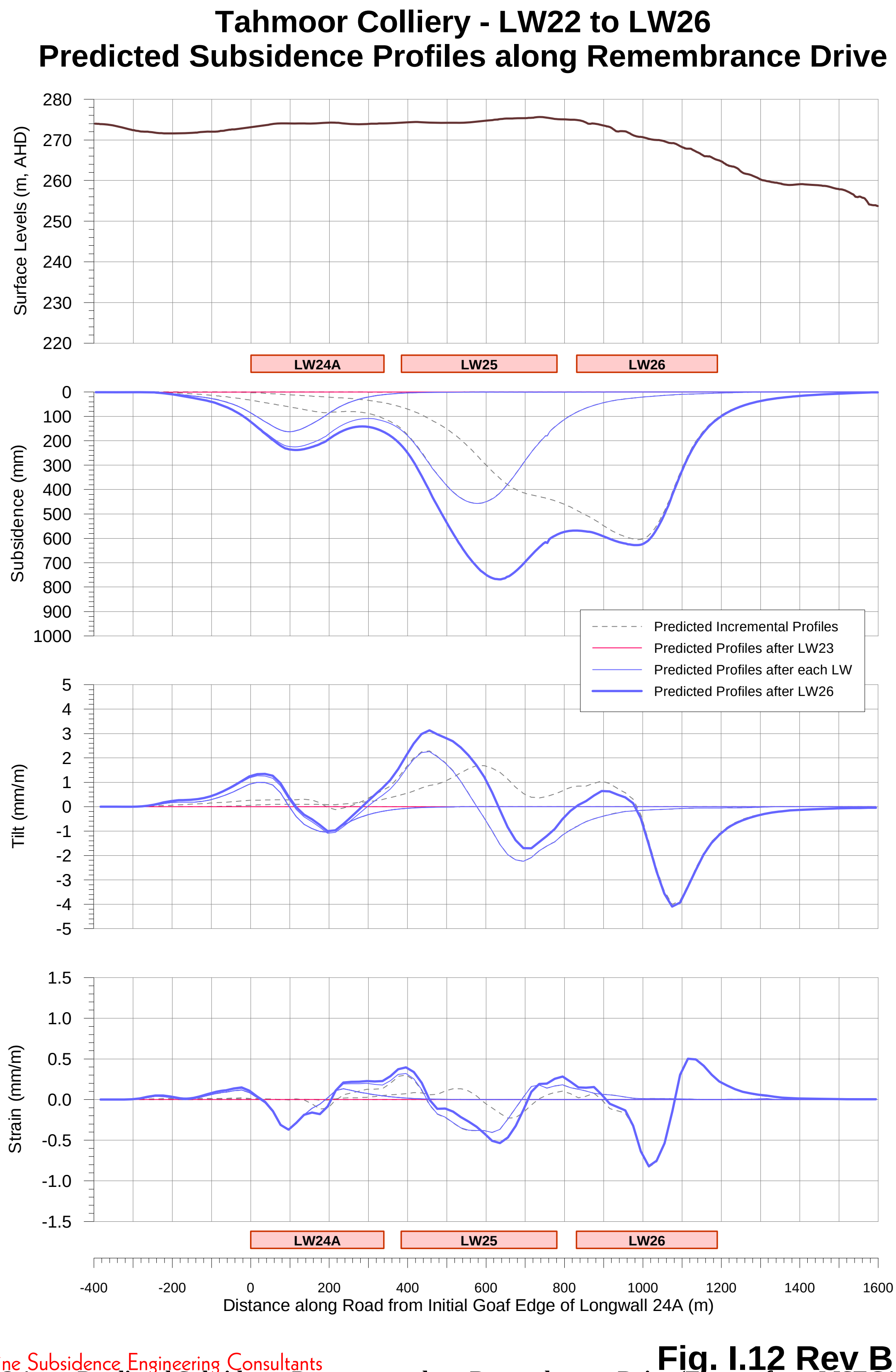


Fig. 3.1 Predicted Subsidence Parameters along Remembrance Drive (Extract from MSEC157)

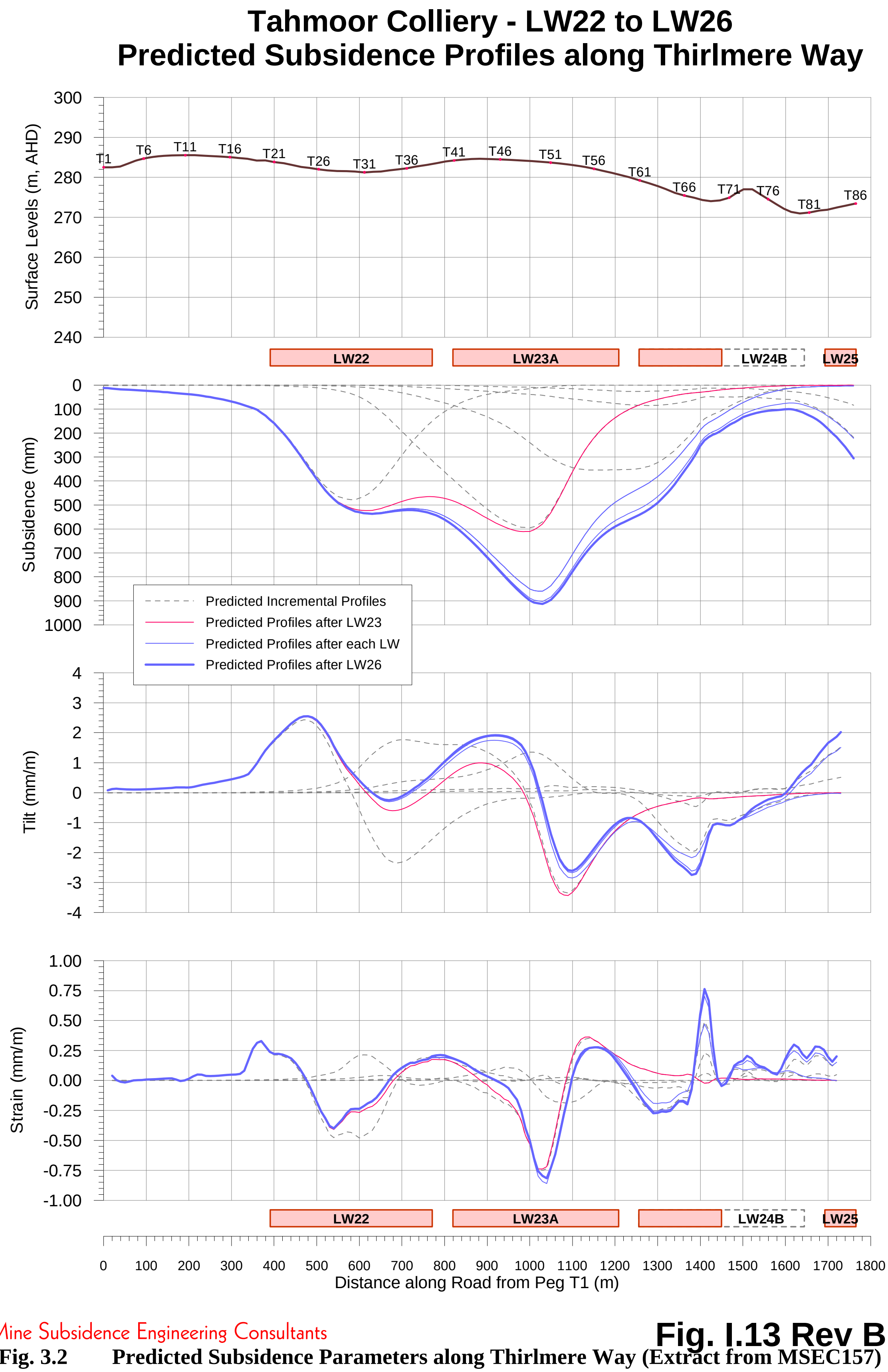


Fig. 3.2 Predicted Subsidence Parameters along Thirlmere Way (Extract from MSEC157)

3.2. Thirlmere Way Overbridge

The Thirlmere Way Railway Overbridge provides the only direct access between the Tahmoor retail and commercial areas and the Tahmoor urban area located west of the Main Southern Railway. This bridge is not owned by Council. The Bridge was strengthened by Tahmoor Colliery prior to the mining of Longwall 24B. A photograph of the bridge is shown in Fig. 3.3.

Fig. 3.3 Thirlmere Way Railway Overbridge

A separate management plan has been developed in consultation with ARTC. Please refer to the separate management plan in Appendix E.

3.3. Bridge on Castlereagh Street over Myrtle Creek

The Castlereagh Street Road Bridge over Myrtle Creek is a two-lane bridge that provides access for residents to Hilton Park Road. The single-span bridge is constructed with a concrete deck on concrete abutments, as shown in Fig. 3.4. This bridge is located above Longwall 25.

Fig. 3.4 Castlereagh Street Road Bridge over Myrtle Creek

3.3.1. Predicted Subsidence Movements

The Castlereagh Street Road Bridge over Myrtle Creek is a two-lane bridge that provides access for residents to Hilton Park Road. This bridge is located above Longwall 25.

Predictions of systematic subsidence, tilt and strain movements have been made at the bridge, and these are shown in Table 3.3.

Table 3.3 Predicted Subsidence Parameters at Castlereagh Street Road Bridge

Stage of Mining	Maximum Predicted Subsidence (mm)	Maximum Predicted Tilt (mm/m)	Maximum Predicted Tension (mm/m)	Maximum Predicted Compression (mm/m)
After Longwall 23	< 20	< 0.1	< 0.1	< 0.1
After Longwall 24	< 20	< 0.1	< 0.1	< 0.1
After Longwall 25	332	4.5	0.7	0.2
After Longwall 26	683	3.6	0.4	0.3

The Bridge will also be subjected to upsidence and closure movements, and these are shown in Table 3.4.

Table 3.4 Prediction of Upsidence and Closure at Castlereagh Street Road Bridge

Stage of Mining	Equiv. Valley Depth (m)	Maximum Cumulative Upsidence (mm)	Maximum Cumulative Closure (mm)
After Longwall 23	6.5	< 5	< 5
After Longwall 24	6.5	15	15
After Longwall 25	6.5	55	38
After Longwall 26	6.5	105	55

The predicted upsidence and closure movements are expected to result in a closure of the sides of the creek valley and a relative upwards movement in the centre of the creek.

Upsidence will have little impact on the bridge as the bridge contains a single span and the relative upwards movements in the bed of the creek cannot be transferred into the bridge deck. However, some differential vertical movements may occur between the abutments of the bridge. It is also possible that upsidence will result in a tilt of each abutment, opening outwards and reducing the closure at the top of the abutment.

While the prediction method for upsidence and closure is considered to be generally conservative, it is possible that the predictions could be exceeded. It is noted, however, that the prediction of closure is based on total closure movements across the full width of the valley. The Bridge span is approximately 12 metres and it is possible that only a proportion of the predicted value will occur between the bridge abutments.

A risk assessment was conducted for the Bridge without consideration of any mitigation measures.

The likelihood of bridge damage and collapse, due to Longwalls 22 to 26, is assessed as **RARE.** The consequence of this risk is assessed as **SEVERE**. The risk is therefore assessed as **RARE / SEVERE → MODERATE**.

The likelihood of the bridge being damaged and requiring repairs, due to Longwalls 22 to 26, is assessed as **UNLIKELY.** The consequence of this risk is assessed as **MODERATE**. The risk is therefore assessed as **UNLIKELY / MODERATE → MODERATE**.

3.3.2. Observed Upsidence and Closure across Myrtle Creek during the mining of Longwall 24B

Tahmoor Colliery has installed a number of monitoring lines across Myrtle Creek and has measured upsidence and closure movements across these lines during the mining of Longwall 24B. Additionally, survey pins were placed on sewer pits on either side of Myrtle Creek at two locations where the sewer pipes crossed, and these were measured on a regular basis (weekly or twice weekly).

The locations of the monitoring lines are shown in Fig. 3.5. The observed movements along Huen Place, Elphin Street, Elphin-Myrtle and Castlereagh Street are shown on following pages in Fig. 3.6, Fig. 3.7, Fig. 3.8, Fig. 3.9 and Fig. 3.10.

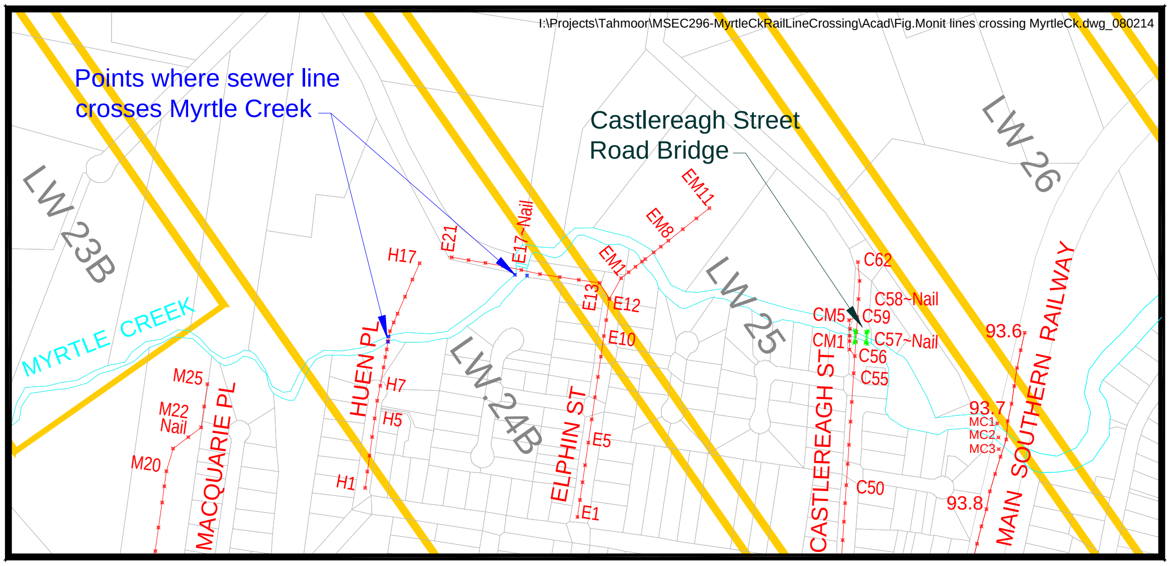


Fig. 3.5 Locations of Survey Marks across Myrtle Creek

A monitoring line was also installed along the Main Southern Railway Corridor along the top of the Myrtle Creek Culvert. Observed ground strains during the mining of Longwall 24B were within survey tolerance.

Three survey marks were installed on the top and base of Myrtle Creek at the Culvert to monitor valley movements during the mining of Longwall 24B (Pegs MC1, MC2 and MC3). A survey was conducted on 6 February 2007, when the length of extraction of Longwall 24B was approximately 770 metres. At this time, all measured strains were within survey tolerance. Unfortunately, the survey marks were destroyed by culvert clearing work shortly afterwards. However, based on the February survey result and consistent results along the railway corridor, it is considered that no measureable upsidence or closure developed at the Myrtle Creek Culvert during the mining of Longwall 24B.

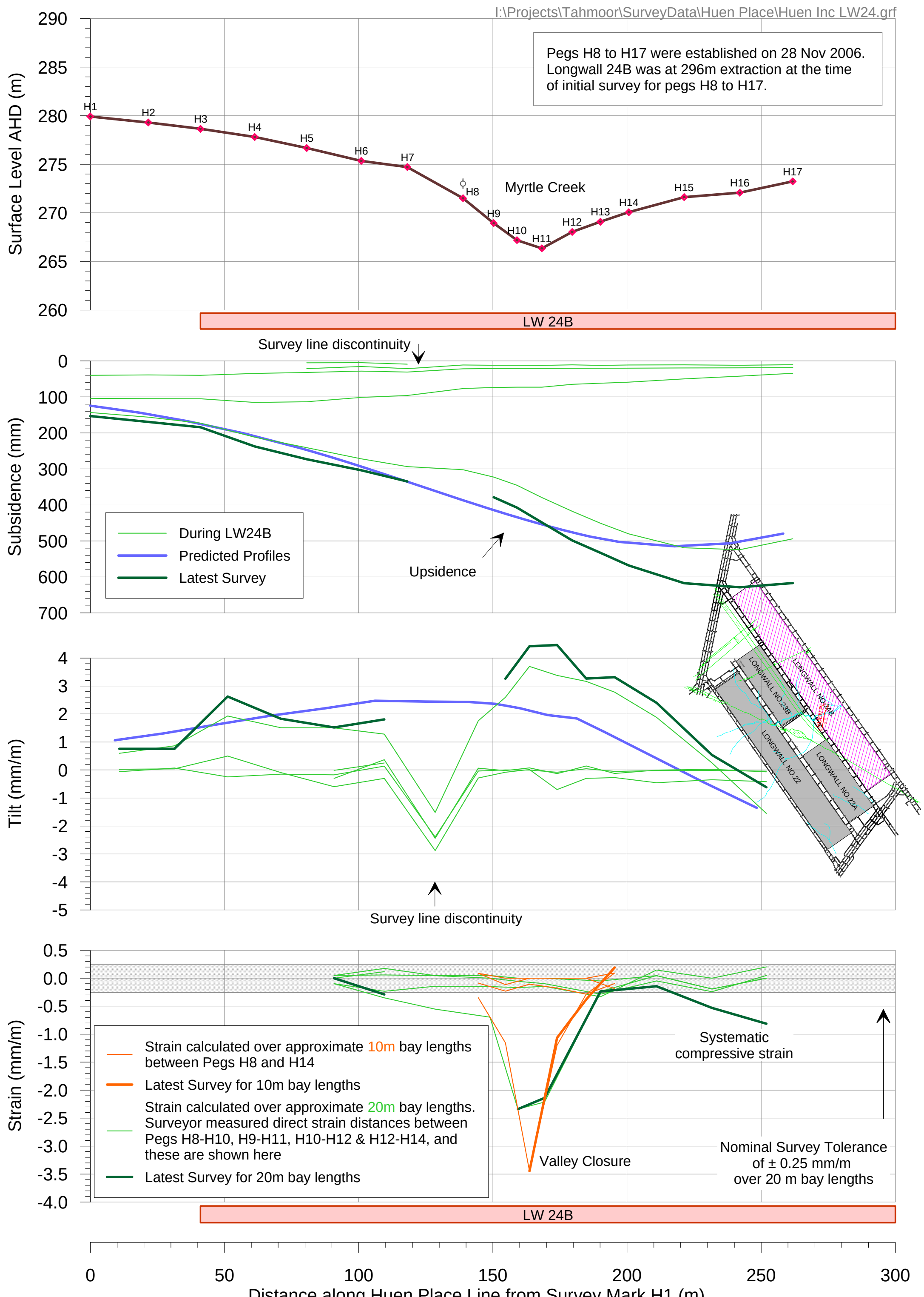


Fig. 3.6 Observed Incremental Movements across Myrtle Creek at Huen Place during Longwall 24B

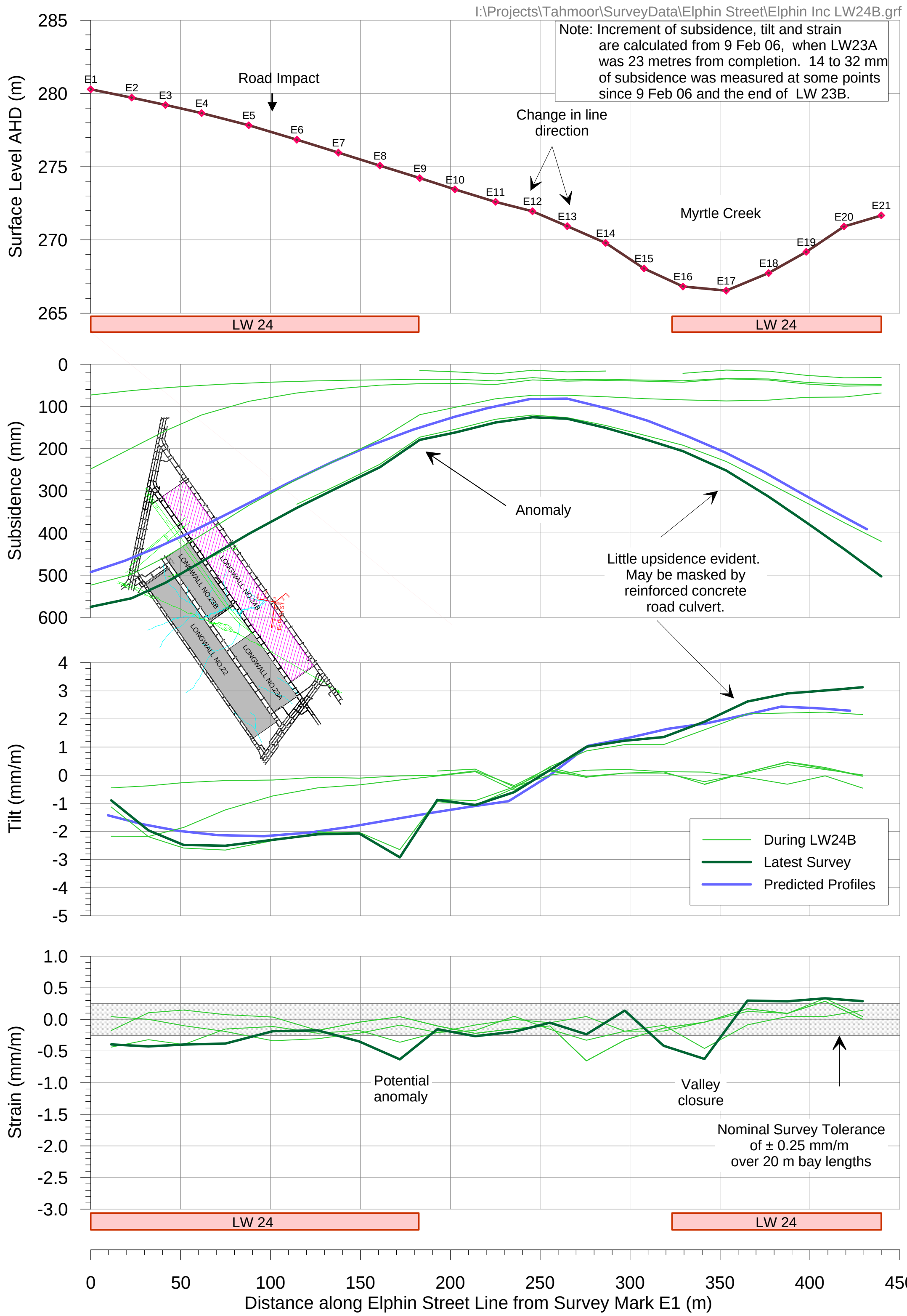


Fig. 3.7 Observed Incremental Movements across Myrtle Creek at Elphin Street during Longwall 24B

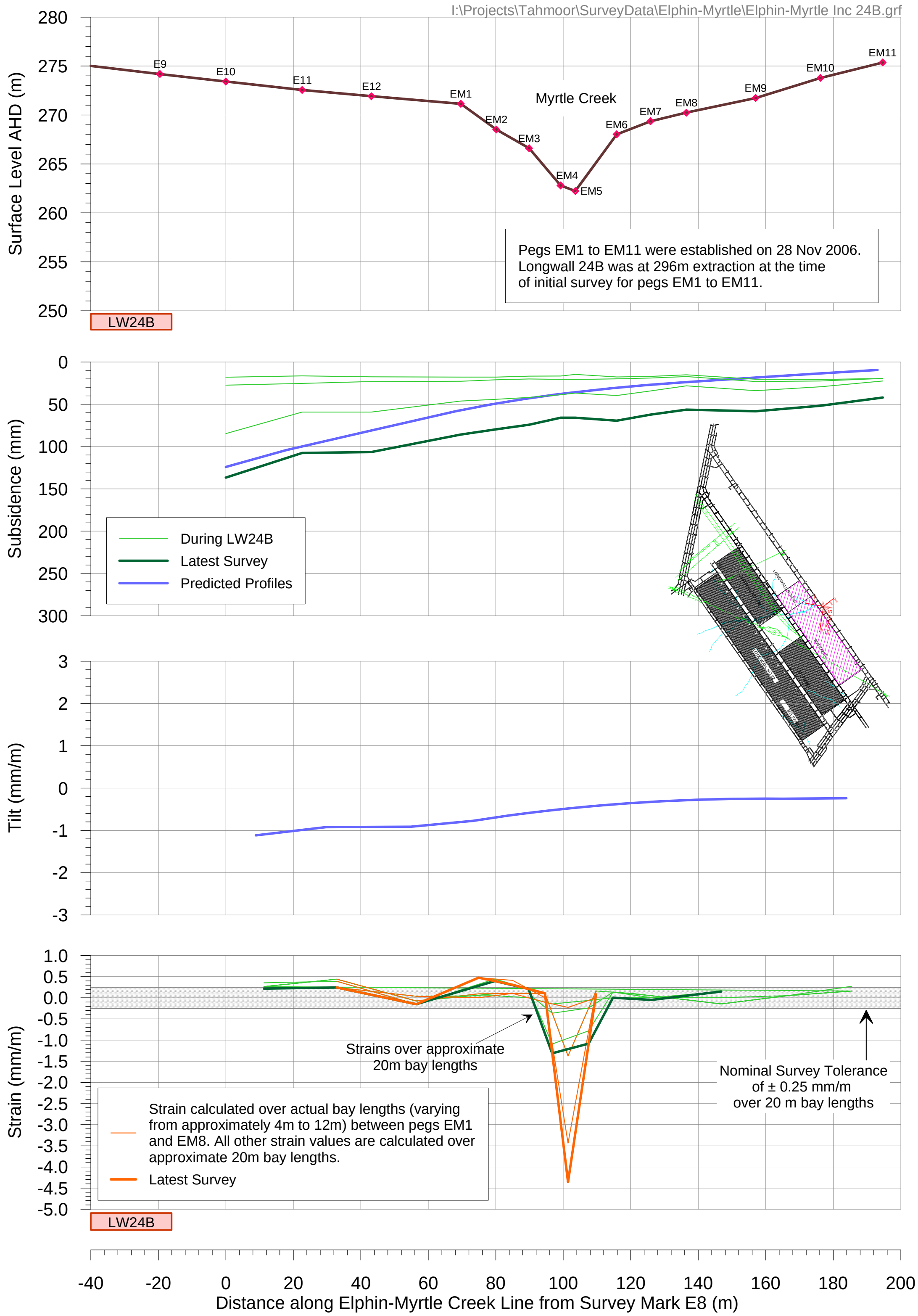


Fig. 3.8 Observed Incremental Movements across Myrtle Creek at Elphin-Myrtle during Longwall 24B

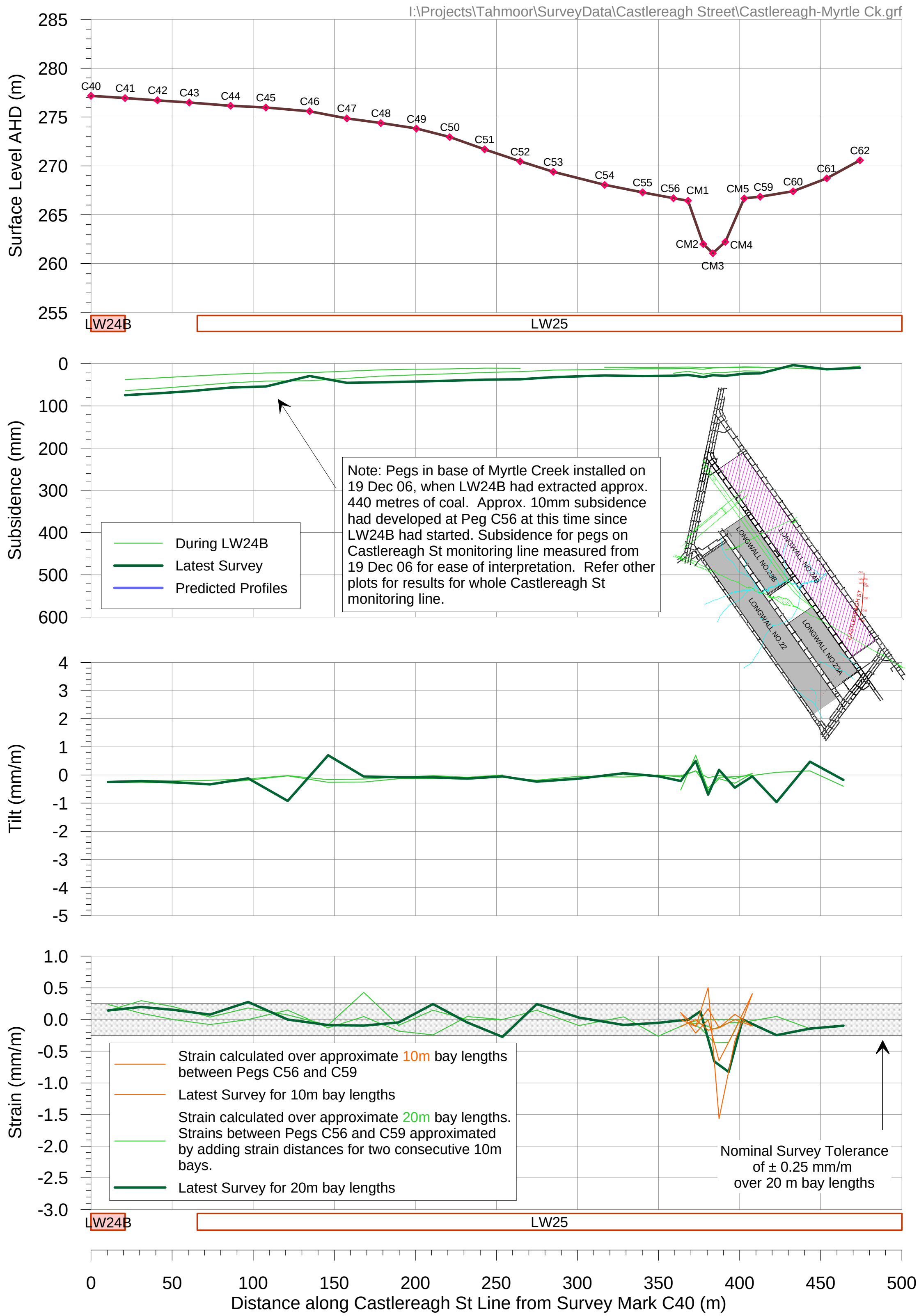


Fig. 3.9 Observed Incremental Movements across Myrtle Creek at Castlereagh St during Longwall 24B

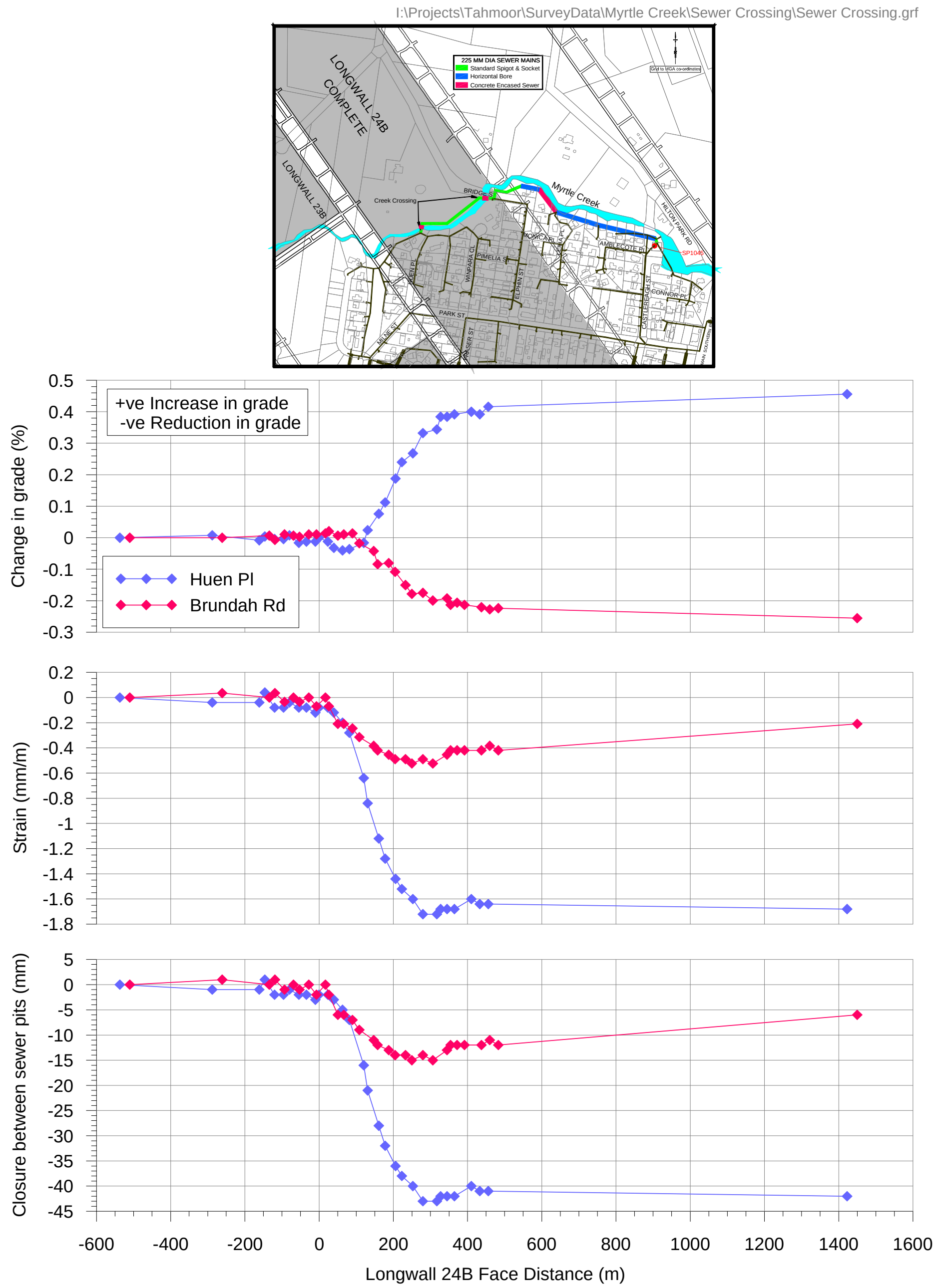


Fig. 3.10 Observed Incremental Strain and Closure along Sewer Pipes that cross Myrtle Creek

Commentary on Observed Upsidence along Myrtle Creek

A summary of observations and comparisons with predicted movements are provided in Table 3.5.

Table 3.5 Observed and Predicted Incremental Upsidence along Myrtle Creek during the extraction of Longwall 24B

Monitoring Line	Observed Incremental Upsidence (mm)	Predicted Incremental Upsidence (mm)	Location of Maximum Incremental Upsidence	Comment
Huen Place	30	50	Near Peg H9, 18 m from creek centre	Upsidence is difficult to measure as the line was extended across Myrtle Creek after LW 24B had commenced, but upsidence bulge is evident
Elphin Street	0	30	N/A	Upsidence may be masked by concrete road culvert. No bending or impact observed in culvert.
Elphin-Myrtle	6	18	Creek centre	-
Castlereagh St	0	10	N/A	-
Myrtle Creek Culvert	<5	5	N/A	No noticeable upsidence along railway corridor monitoring line. Pins were placed at base of creek. First survey at 770 m of extraction indicated some upsidence (< 5 mm) but within survey tolerance. Pegs were lost following this survey.

In this case, it can be seen from the results that actual upsidence has been less than predicted for all survey lines across Myrtle Creek. No measurable upsidence was observed at Myrtle Creek Culvert during the mining of Longwall 24B.

It is interesting to observe that no upsidence was noticeable along Elphin Street where it crosses Myrtle Creek. The survey pegs are located on top of a reinforced concrete culvert, which may have masked any upsidence that might have developed. Photographs of the road culvert are shown below.

Fig. 3.11 Photographs of Elphin Street Road Culvert at Myrtle Creek

Commentary on Observed Incremental Closure along Myrtle Creek

A summary of observations and comparisons with predicted incremental movements are provided in Table 3.6.

Table 3.6 Observed and Predicted Incremental Closure along Myrtle Creek during the extraction of Longwall 24B

Monitoring Line	Observed Incremental Closure (mm)	Predicted Incremental Closure (mm)	Maximum Observed Incremental Compressive Strain (mm/m)	Location of Maximum Strain	Comment
Huen Place	87	24	2.3 (18 m) 3.4 (9.3 m)	Creek centre	32 mm closure over 9.3 m bay 42 mm closure over 25 m bay between sewer pits Monitoring has experienced systematic compression strains in addition to closure strains.
Elphin Street	30	33	0.7 (24 m)	Creek centre	18 mm closure over 24 m bay Up to 15 mm closure over 29 m bay between sewer pits though final closure was 6 mm
Elphin-Myrtle	20	24	1.5 (14 m) 4.6 (4.4 m)	Creek centre	20 mm closure over 4.4 m bay
Castlereagh St	22	12	0.8 (19 m) 1.6 (7.7 m)	Creek centre	12 mm closure over 7.7 m bay
Myrtle Creek Culvert	<5	5	<0.3	Too small to be certain	No noticeable closure along railway corridor monitoring line. Pins were placed at base of creek. First survey at 770 m of extraction indicated some closure (3 mm) but within survey tolerance. Pegs were lost following this survey.

While observed upsidence was less than predicted, the observed closure has exceeded predictions in two locations along Myrtle Creek. The most noticeable site is Huen Place, where 87 mm of closure has developed when only 24 mm was predicted. However, it should be noted that the observed closure along this Huen Place monitoring line includes a large component of systematic compressive strain, which is difficult to extract from the observed data.

Unlike the other sites, Huen Place is located in an area of systematic compressive strain and calculations undertaken recently show that the total closure from the systematic strains along other monitoring lines crossing LW24B where there are not have creeks, i.e. Castlereagh, Park and Chapman Road monitoring lines, accumulated to 50 to 80 mm. Additionally, it can be seen in Fig. 3.6 that compressive strains concentrate at the Creek but also increased near the end of the monitoring line where systematic compressive strains are expected. Therefore the actual valley closure component of the observed closure at Huen Place is likely to be small, but, it is difficult to attribute the components of closure accurately.

It can be seen from each of these monitoring results that the closure profile is distributed across Myrtle Creek in different ways. At Huen Place, approximately 32 mm of the total valley closure of 87 mm developed across a distance of approximately 9.3 metres. In contrast, observed closure at Elphin-Myrtle Creek is concentrated solely over a distance of 4.4 metres in the base of the creek. This is understandable as systematic tensile strains were expected to develop along this monitoring line as a result of systematic subsidence, with compressive closure strain only observed in the centre of the Creek.

Predicted closure is generally conservative but can be exceeded on occasion. In this case, it can be seen from the results that actual closure has been exceeded at two of five locations across Myrtle Creek, although Huen Place has also experienced systematic compressive strain.

The other monitoring line where a small exceedence in observed over predicted closure was measured is the Castlereagh St Monitoring Line, as shown in Fig. 3.9. As detailed in Table 3.6 a closure of 22 mm was observed when the predicted closure was 12 mm. Systematic tensile strains were expected at this location. It is noted that 12 mm of closure was observed across a 7.7 metre survey bay.

No measurable closure was observed at Myrtle Creek Culvert during the mining of Longwall 24B.

3.3.3. Observed Subsidence Movements at Castlereagh Street Bridge during the mining of Longwalls 23A to 24B

Survey marks were installed on the bridge deck and abutment prior to the influence of Longwall 23A. Observed subsidence following the completion of Longwall 24B was 57 mm. This is slightly greater subsidence than predicted but the difference is very small. Observed tilts and strains are currently within survey tolerance.

Measured closure between the bridge abutments following the completion of Longwall 24B was 3 mm, which is within survey tolerance and less than the predicted closure of 15 mm.

The measured abutment closure movements of 3 mm are less than the measured ground closure movements across the creek, which were measured to be 12 mm across a 7.7 metre survey bay in the base of the creek, and 22 mm across the whole valley. This suggests that the Bridge is currently resisting the small ground movements that have occurred.

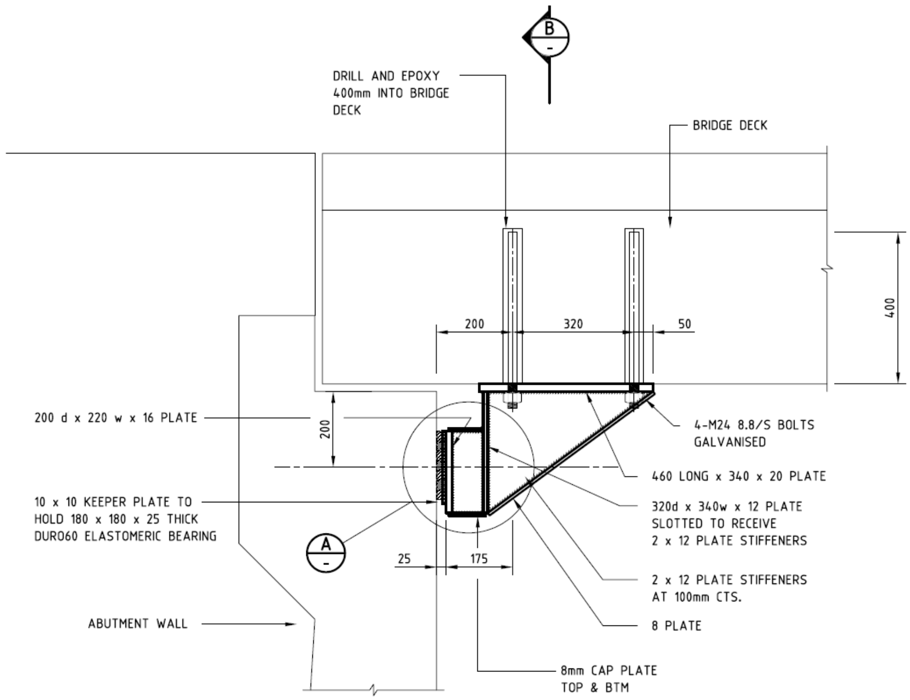
3.3.4. Structural Assessment and Design of Mitigation Measure

The single-span bridge is constructed with a concrete deck on concrete abutments. The span of the deck is approximately 12 metres and the heights of the abutments are approximately 6 and 8 metres high.

The deck comprises pre-tensioned bridge units that have been integrated with a reinforced concrete slab. The reinforced concrete abutments and wing walls have been dowelled and grouted into the bedrock. The bridge deck rests on the abutments with rubber bearing pads and is fixed at both ends with vertical dowelled joints spaced at regular centres. There is approximately 5 mm of clearance between each dowel and its hole and a rubber ring has been placed in this gap. Each abutment includes a small upstand that prevents the deck from sliding. There is a 20 mm gap between the upstand and the ends of the deck. The gap is filled with polystyrene fillers, rubber buffers and mastic fillers.

A structural assessment of Castlereagh Street Bridge was undertaken by John Matheson & Associates (JMA, 2006), based on a site inspection and review of structural drawings that were provided by Wollondilly Shire Council. The assessment identified that in the event of abutment closure, it is expected that the concrete hob behind the abutment will initially crack and shear. This impact is not expected to result in any reduction in stability of the bridge. As abutment closure increases, the steel dowels that connect the deck to the abutment walls are expected to shear. This impact is not expected to result in any long-term structural concerns as the abutment walls were designed as freestanding cantilevers. However, JMA has identified a potential risk where cracking may occur to the abutment corbel at the time the dowels shear and that this cracking may occur suddenly as the abutment support transfers from a propped cantilever to a freestanding cantilever.

JMA recommended that the abutments could be protected at the moment of dowel failure by the installation of steel brackets bolted to the end of each bridge deck girder (13 number girders and 26 number brackets), which bear against the abutment walls (JMA, 2007). The brackets provide additional support to the abutment walls, ensuring that the dowels shear without breaking the concrete that surround them. The design of the brackets is shown in Fig. 3.12 and a photograph of the completed installation is shown in Fig. 3.13. The galvanised brackets were installed by Tahmoor Colliery prior to the commencement of Longwall 25.



Extract of drawing by JMA (Drawing No. 0622-1.00)

Fig. 3.12 Design of Mitigation Measures beneath Bridge Deck

Photograph courtesy of John Matheson & Associates

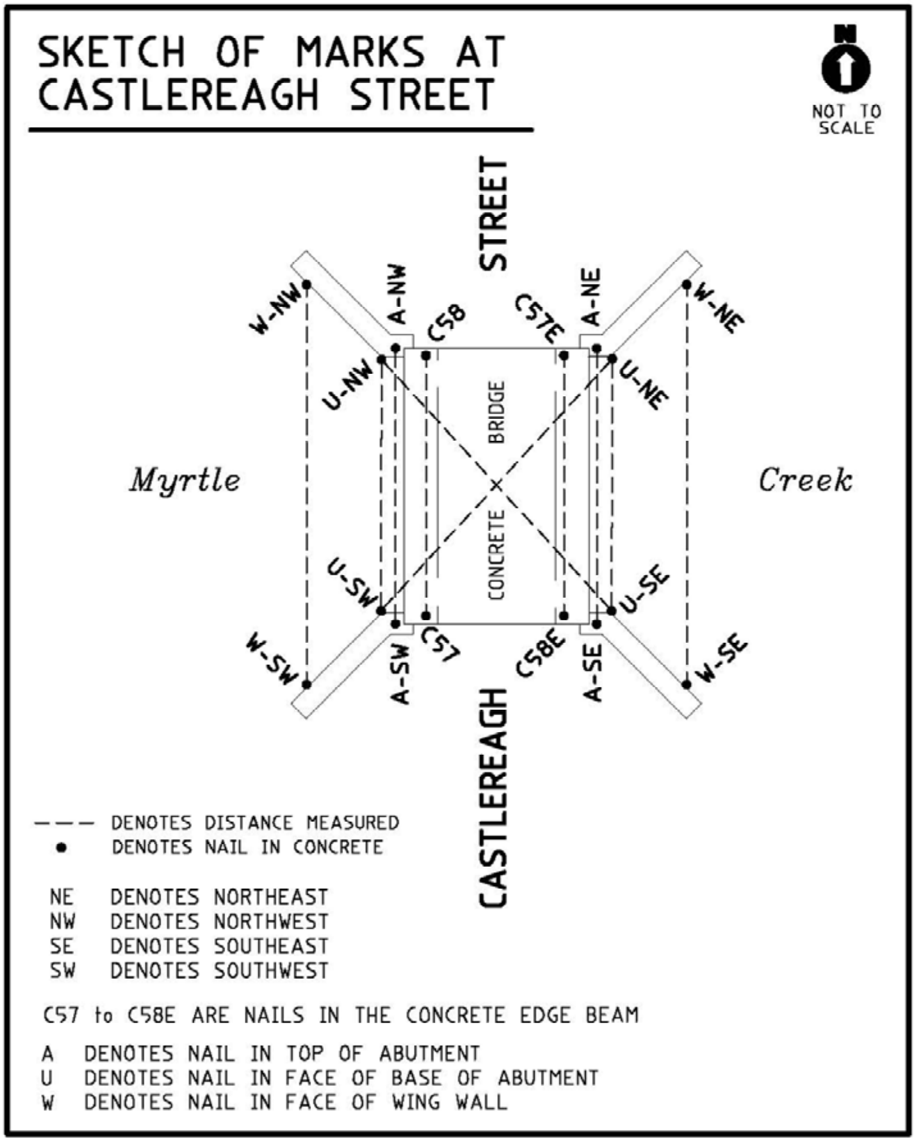
Fig. 3.13 Photograph of Installed Brackets at Castlereagh Street Bridge

It is further noted that the road pavement may spall and buckle at the abutments as a result of valley closure movements.

3.3.5. Monitoring measures

The Castlereagh Street Bridge will be monitored and inspected in three ways:

- Structure survey
  Survey marks have been placed on the top and bottom of the abutment walls, and at the bottom of the wing walls. Marks have also been placed on the bridge deck at each end. The survey marks were installed prior to the influence of Longwall 23A. A sketch of the monitoring mark locations is shown in Fig. 3.14.



Sketch courtesy of Lean & Hayward Surveyors

Fig. 3.14 Location of Survey Marks on Castlereagh Street Bridge

- Bridge Abutment Survey
  Surveys will be conducted to measure differential horizontal movement between the bridge abutment and bridge deck.

- Street survey along Castlereagh Street
  Survey marks have been placed along Castlereagh Street on either side of the Bridge. The street survey will provide general information on subsidence and overall valley closure movements.
- Survey across Myrtle Creek adjacent to Bridge
  Survey marks have been placed on the upstream side of Castlereagh Street Bridge to measure valley ground movements adjacent to the Bridge. This will provide information about whether the in Myrtle Creek on the top and bottom of the abutment walls, and at the bottom of the wing walls. Marks have also been placed on the bridge deck at each end. The survey marks were installed prior to the influence of Longwall 24B.
- Visual Inspections
  A qualified building inspector will undertake visual inspections of the Bridge and approaches during mining and report any signs of impact.

3.3.6. Triggers and Responses

JMA (2008) has provided advice regarding response measures that can be implemented if triggered by monitoring results. A three stage trigger process will be adopted in relation to the Bridge where the level of response increases for increasing trigger levels.

Table 3.7 Trigger Levels for Castlereagh Street Bridge

Trigger Level	Cracking to Abutment Walls	Abutment Movement relative to Bridge Deck	Triggered Response
GREEN	Cat 0 crack width <0.3mm	< 20 mm movement between abutment wall and bridge deck	No response other than standard monitoring and inspection procedures.
BLUE)	Cat 1 single cracks 0.3 mm to 1.0mm spaced at approx. 500 mm centres	20 mm total movement comprising 10 mm movement at each abutment wall due to bearing compression	Consider the following measures: - increase monitoring frequency - structural inspection of Bridge - install crack gauges and measure growth - adjust position of galvanised steel brackets or remove brackets
YELLOW	Cat 2 single cracks 1.0 mm to 5.0mm spaced at approx. 500 mm centres	If brackets are still in place: 30 mm total movement comprising 15 mm movement at each abutment wall due to bearing compression	Consider the following measures: - increase monitoring frequency - structural inspection of Bridge - install temporary props to strut the abutment walls to reduce earth pressure
ORANGE	Cat 3 single cracks of width > 5.0 mm spaced at approx. 500 mm centres	If brackets are still in place: 60 mm total movement comprising 30 mm movement at each abutment wall due to bearing compression	Consider the following measures: - increase monitoring frequency - structural inspection of Bridge - install temporary props to strut the abutment walls to reduce earth pressure - Limit bridge traffic

The above triggers vary depending on whether the steel brackets are left in place after they have achieved their design purpose of preventing concrete blow out when the steel dowels shear.

If abutment closure develops significantly, it is possible that the brackets, after achieving their design purpose of preventing abutment cracking at time of dowel shear, may reach the limit of their bearings and begin to apply lateral loads onto the abutment walls.

The BLUE trigger for abutment movement has been determined by the closure at which it is considered that the dowels have sheared. The YELLOW trigger for abutment movement has been determined by the closure at which it is considered that the steel brackets are directly touching the abutment wall and rigid body movements have begun. The ORANGE trigger for abutment movement has been determined by calculated loads on the abutment walls if the brackets have not been removed.

A decision on whether to adjust or remove brackets can be made during the active subsidence period based on actual observations. It will be necessary to temporarily prop the abutments when removing the brackets, as the brackets and abutment walls will be under load. Once the brackets are removed, the jacking force in the props can be gradually reduced. It is likely that brackets will only need to be removed on one abutment only.

3.4. Culverts

There are many culverts within the SMP Area. Twenty-seven (27) culverts have been identified within the SMP Area that carry water from creeks and watercourses under local roads and railways, as shown in Drawing No. MSEC286-040202. Further details regarding these culverts are provided in Table C.1.

The Bridge Street (Tahmoor) Road Culvert over Myrtle Creek (Ref. C30) supports a two-lane road that connects Brundah Road in Thirlmere with Elphin Street in Tahmoor. The concrete culvert consists of two 1500 mm diameter pipes and concrete abutments. The culvert is located above Longwall 24B.

The Turner Street Road Culvert over Redbank Creek (Ref. C16) supports a two-lane road. The concrete culvert consists of four 1500 mm square sections and concrete abutments. The culvert lies outside the predicted limit of subsidence.

There is a small concrete culvert over Myrtle Creek (Ref. C40), which provides vehicular access to Property Y58 from York Street, Tahmoor. The single lane low-level crossing consists of two 600 mm diameter pipes, which have been encased in concrete. The culvert is located above Longwall 26.

The remaining culverts are generally small in size, and typically range between 450 mm and 900 mm in diameter. Predictions of systematic subsidence, tilt and strain are provided in Table C.1.

A summary of the maximum predicted subsidence, tilts and strains at the larger road culverts are provided in Table 3.8. This table also includes a summary of the maximum subsidence, tilt and strain at all culverts within the SMP Area. The maximum predicted tilts and strains are the maximum predictions of tilt during and after each of the proposed longwalls.

Table 3.8 Maximum Predicted Subsidence Parameters at Road Culverts

Culvert Reference	Location	Size (mm)	Maximum Predicted Subsidence (mm)	Maximum Predicted Tilt (mm/m)	Maximum Predicted Tension (mm/m)	Maximum Predicted Compression (mm/m)
C16	Turner Street	4 x 1500 square	< 20	< 0.1	< 0.1	< 0.1
C30	Bridge Street	2 x ϕ1500	647	1.4	0.5	0.3
C40	Property Y58	2 x ϕ600	623	0.6	0.5	0.5
All other Culverts	SMP Area	-	692	4.7	0.6	0.9

The culverts will be subjected to travelling tilts and strains due to the subsidence waves that move through as each longwall face passes beneath the road. These travelling tilts and strains are generally aligned along the longitudinal axes of the longwalls with the maximum values generally occurring in the locations of maximum incremental subsidence for each longwall.

The culverts are also expected to experience closure and upsidence due to the extraction of Longwalls 25 to 26.

Predictions of closure, upsidence and compressive strain due to closure at the drainage culverts due to the extraction of Longwalls 22 to 26 are shown in Table C.1. The method of prediction is described in Report No. MSEC157.

A summary of the maximum predicted upsidence, closure and compressive strain due to closure at the larger road culverts are provided in Table 3.9. This table also includes a summary of the maximum upsidence, closure and compressive strain due to closure at all culverts within the SMP Area.

Table 3.9 Maximum Predicted Closure and Upsidence at the Road Culverts

Culvert Reference	Location	Equiv. Valley Depth (m)	Maximum Cumulative Upsidence (mm)	Maximum Cumulative Closure (mm)	Maximum Compressive Strain (mm/m)
C16	Turner Street	4.0	22	8	< 1.0
C30	Bridge Street	8.5	114	88	7.4
C40	Property Y58	4.5	47	32	3.3
All other Culverts	SMP Area	Varies	48	34	3.5

It is expected that the subsidence induced tilts will not significantly affect the drainage flows in the culverts as the changes in grade are all less than 0.5%. The maximum predicted systematic strains are also very small and unlikely to damage the culverts.

The Turner Street Culvert (Ref. C16) is located outside the predicted limit of subsidence and will experience small movements of upsidence, closure and compressive strain only. It is unlikely, therefore, that the culvert will be adversely impacted by the proposed longwalls. If any impacts occur, they are more likely to appear in the road surface. No impacts were observed at the culvert following the mining of Longwall 24B, which is the closest longwall to the culvert.

The Bridge Street Culvert (Ref. C30) is predicted to experience 114 mm of upsidence. Upsidence is unlikely to adversely impact the culvert as it is expected to occur along its entire length. However, it is possible that the road surface will slightly bulge near the centre of the culvert, and may require resurfacing. The predicted compressive strains associated with closure and upsidence at the culvert are 7.4 mm/m. While these strains are large enough to cause cracking in concrete, they are unlikely to adversely impact the Bridge Street Culvert as they will be orientated perpendicular to the culvert across its strongest axis. If any impacts occur, they are more likely to appear in the road surface or in the concrete headwalls. Any impacts will occur gradually as mining progresses, which will provide adequate time to repair the road at appropriate stages to maintain the safe operation of the road. Longwall 24B directly mined beneath the culvert and no impacts were observed. Please refer to Section 3.3.2 for information on ground surveys undertaken near the Culvert.

The culvert at Myrtle Creek at Property Y58 (Ref. C40) is predicted to experience 47 mm of upsidence. Upsidence is unlikely to adversely impact the culvert as it is expected to occur along its entire length. However, it is possible that the road surface will slightly rise near the centre of the culvert, and a small step may form at the joint between the culvert and driveway pavement. The predicted maximum compressive strain associated with closure and upsidence in the culvert is 3.3 mm/m. While this strain is large enough to cause cracking in concrete, it is unlikely to adversely impact the culvert as it will be orientated perpendicular to the culvert across its strongest axis. It is recommended that the culvert be visually monitored during mining. Any impacts will occur gradually as mining progresses, which will provide adequate time to repair the road surface at appropriate stages to maintain the safe operation of the access road.

The risk of impacts to smaller culverts is considered low. No impacts to culverts have been reported during the mining of Longwalls 22 to 24.

The hazard associated with culverts is that they could be damaged and/or rendered unserviceable from mine subsidence impacts.

The likelihood of extensive damage is assessed as **VERY RARE**. The consequence of this risk is assessed as **MODERATE**. The risk is therefore assessed as **VERY RARE / MODERATE → LOW.**

The likelihood of minor damage is assessed as **UNLIKELY**. The consequence of this risk is assessed as **SLIGHT**. The risk is therefore assessed as **UNLIKELY / SLIGHT → LOW.**

CHAPTER 4. RISK CONTROL PROCEDURES

4.1. Structures Response Group (SMG)

The SMG is responsible for taking the necessary actions required to manage the risks that are identified from monitoring of structures. The SMG's key members are:

- Tahmoor Colliery
- Wollondilly Shire Council
- John Matheson and Associates
- Mine Subsidence Engineering Consultants
- Mine Subsidence Board
- Sunrise Building and Property Services

4.2. Mitigation Measures

Mitigation measures have been undertaken for Castlereagh Street Bridge, as described in Section 3.3.4.

4.3. Monitoring Measures

Monitoring lines have been installed along all streets within the urban area above Longwalls 25 and 26, as shown in Fig. 4.1. The monitoring lines have been initially surveyed to provide a baseline reference. Monitoring of street survey lines will be conducted for every 200 metres of longwall travel as a minimum for pegs located within the active subsidence zone.

Additional surveys will be conducted for the Castlereagh Street Bridge and Bridge Street culvert over Myrtle Creek, as described in Table 4.1.

4.4. Risk Control Procedures

Risk control procedures are provided in Table 4.1. The procedures include responses if triggered by monitoring results.

In relation to the Castlereagh Street Bridge, please refer to Section 3.3.6.

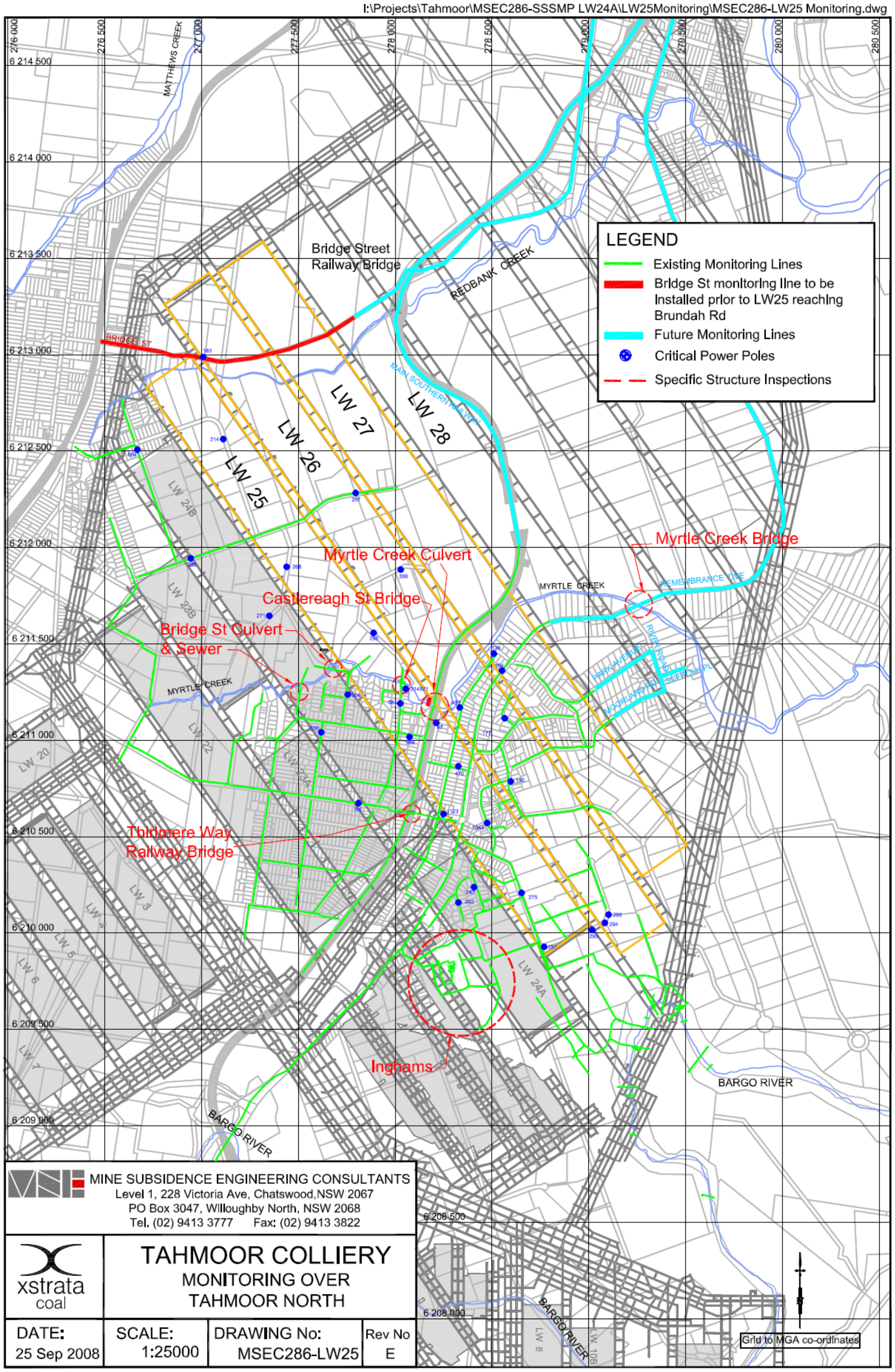


Fig. 4.1 Monitoring Lines above Longwall 25

Table 4.1 Risk Control Procedures

Infrastructure	Hazard / Impact	Risk	Trigger	Control Procedure/s	Frequency	By Whom?
Local Roads	Impacts to Roads Impacts to Culverts	**MODERATE**	None	Conduct visual inspection for surface deformations along Thirlmere Way and Remembrance Drive	Twice a week when the roads are within active subsidence area	Tahmoor Colliery (SBPS)
				Conduct surveys along survey lines to provide some early warning for potentially damaging subsidence events	Every 200 metres of longwall face movement	Tahmoor Colliery (L&H/MSEC)
			Impacts occur	Notify all stakeholders, including Council, Tahmoor Colliery, Mine Subsidence Board and Department of Primary Industries – Minerals	Within 24 hours	Tahmoor Colliery / WSC
				Repair road	As required	WSC
Thirlmere Way Overbridge				Please refer to the Management Plan for the Thirlmere Way Overbridge in Appendix E		
Culvert on Bridge Street over Myrtle Creek	Subsidence and Closure	**LOW**	None	Install survey marks to measure subsidence and closure.	Complete	Tahmoor Colliery (L&H)
				Conduct visual inspections of culvert.	Twice a week when the bridge is within active subsidence area.	Tahmoor Colliery (SBPS)
				Conduct surveys of culvert during mining	Every 200 m of longwall travel when culvert is within active subsidence zone and End of LWs 25 & 26	Tahmoor Colliery (L&H)
			Impacts occur to culvert	Notify all stakeholders, including Council, Tahmoor Colliery, Mine Subsidence Board and Department of Primary Industries – Minerals	Within 24 hours	Tahmoor Colliery / WSC
				Consider potential future impacts and implement risk control procedures	As required	Tahmoor Colliery / WSC

Infrastructure	Hazard / Impact	Risk	Trigger	Control Procedure/s	Frequency	By Whom?
Bridge on Castlereagh Street over Myrtle Creek	Subsidence and Closure	MODERATE	**GREEN** (None)	Install survey marks to measure subsidence and closure.	Complete	Tahmoor Colliery (L&H)
				Conduct baseline monitoring of position of abutment wall relative to bridge deck	Prior to LW25 approaching within 600 metres of Bridge (1000m of extraction)	Tahmoor Colliery (SBPS)
				Install mitigation measures	Complete	Tahmoor Colliery
				Arrange contractor on standby for swift adjustment or removal of brackets after dowel shear	Prior to LW25 approaching within 600 metres of Bridge	Tahmoor Colliery
				Conduct surveys at Bridge, including: • Structure surveys • Castlereagh Street ground survey (all pegs within 80 metres of Bridge – Pegs C54 to C62) • Myrtle Creek cross line adjacent to Castlereagh Street Bridge)	Weekly during LWs 25&26 when longwall face has approached within 200 m of Bridge until agreed to reduce	Tahmoor Colliery (L&H)
				Conduct visual inspections of Bridge (including bridge abutment surveys)	For LW25: Weekly once LW25 is within 200 metres of Bridge Daily once LW25 is within 50 metres of Bridge until agreed to reduce plus end of LW For LW26: Weekly within active subsidence zone until agreed to reduce plus end of LW	Tahmoor Colliery (SBPS)
				Assess monitoring results and report	Weekly when bridge within active subsidence zone until agreed to reduce	Tahmoor Colliery (MSEC)
			BLUE (refer Table 3.7)	Convene meeting of SMG to consider additional monitoring and mitigation measures based on observed monitoring results, which may include: • Increase monitoring and inspection frequency • Structural inspection of Bridge • Install crack gauges and measure growth • Adjust position or remove galvanised steel brackets (temporary propping required)	Within 48 hours	SMG
			YELLOW (refer Table 3.7)	Convene meeting of SMG to consider additional monitoring and mitigation measures based on observed monitoring results, which may include: • Increase monitoring and inspection frequency • Structural inspection of Bridge • Install temporary props to strut the abutment walls to reduce earth pressure Note: Trigger for abutment closure only relevant if brackets are still in place	Within 48 hours	SMG
			ORANGE (refer Table 3.7)	Convene meeting of SMG to consider additional monitoring and mitigation measures based on observed monitoring results, which may include: • Increase monitoring and inspection frequency • Structural inspection of Bridge • Install temporary props to strut the abutment walls to reduce earth pressure • Limit bridge traffic Note: Trigger for abutment closure only relevant if brackets are still in place	Within 24 hours	SMG
			Impacts occur to Bridge or approaches	Notify all stakeholders, including Council, Tahmoor Colliery, Mine Subsidence Board and Department of Primary Industries – Minerals	Within 24 hours	Tahmoor Colliery / WSC
				Consider potential future impacts and implement risk control procedures	As required	SMG

CHAPTER 5. SMG MEETINGS

The monitoring of natural surface features and surface infrastructure which forms an integral part of this Management Plan will be carried out by Tahmoor Colliery. SMG Meetings will be held between Tahmoor Colliery, Wollondilly Shire Council, the Mine Subsidence Board and / or the Department of Mineral Resources for discussion and resolution of issues raised in the operation of the Management Plan. The frequency of the SMG Meetings will be monthly unless agreed otherwise between SMG members.

A secretary will be appointed at the SMG Meeting. All documentation, distribution of meeting minutes and organising of meeting times will be undertaken by the secretary.

SMG Meetings will discuss any incidents reported in relation to the relevant surface feature, the progress of mining, the degree of mine subsidence that has occurred, and comparisons between observed and predicted ground movements.

It will be the responsibility of the meeting representatives to determine whether the incidents reported are due to the impacts of mine subsidence, and what action will be taken in response.

In the event that a significant risk is identified for a particular surface feature, any party may call an emergency SMG Meeting, with one day's notice, to discuss proposed actions and to keep other parties informed of developments in the monitoring of the surface feature.

CHAPTER 6. AUDIT AND REVIEW

All Management Plans within this document have been agreed between parties. The Management Plan will be reviewed following extraction of each longwall.

Should an audit of the Management Plan be required during that period, an auditor shall be appointed by the Tahmoor Colliery to review the operation of the Management Plan and report at the next scheduled Plan Review Meeting.

Other factors that may require a review of the Management Plan are:-

- Observation of greater impacts on surface features due to mine subsidence than was previously expected.
- Observation of fewer impacts or no impacts on surface features due to mine subsidence than was previously expected.
- Observation of significant variation between observed and predicted subsidence.

CHAPTER 7. RECORD KEEPING

The secretary will keep and distribute regular minutes of each Plan Review Meeting for each surface feature. The minutes will include reports on the condition of the relevant surface feature, the progress of mining, the degree of mine subsidence that has occurred, comparisons between observed and predicted ground movements, agreements reached between parties, and a log of incidents that have occurred on the surface feature.

CHAPTER 8. CONTACT LIST

Organisation	Contact	Phone	Email / Mail	Fax
Department Primary Industries (Mineral Resources Division)	Phil Steuart	(02) 4931 6648	phil.steuart@dpi.nsw.gov.au	(02) 4931 6790
Department Primary Industries (Mineral Resources Division)	Gang Li	(02) 4931 6644 0409 227 986	gang.li@dpi.nsw.gov.au	(02) 4931 6790
Department Primary Industries (Mineral Resources Division)	Ray Ramage	(02) 4931 6645 0402 477 620	ray.ramage@dpi.nsw.gov.au	(02) 4931 6790
John Matheson & Associates (JMA)	John Matheson	(02) 9979 6618	jma.eng@bigpond.net.au	(02) 9999 0121
Mine Subsidence Board	Darren Bullock	(02) 4677 1967	d.bullock@minesub.nsw.gov.au	(02) 4677 2040
Mine Subsidence Engineering Consultants (MSEC)	Daryl Kay	(02) 9413 3777	daryl@minesubsidence.com	(02) 9413 3822
Sunrise Building and Property Services (SBPS)	John Schwarz	(02) 4883 9030 0400 390058	sunbuilding@westnet.com.au	(02) 4883 9738
Tahmoor Colliery	David Clarkson	(02) 4640 0133	d.clarkson@xstratacoal.com.au	(02) 4640 0140
Tahmoor Colliery (Senior Mine Surveyor)	Mark Rundle	(02) 4640 0155	m.rundle@xstratacoal.com.au	(02) 4640 0140
Wollondilly Shire Council Manager - Works	Justin Nyholm	(02) 4677 8247	justin.nyholm@wollondilly.nsw.gov.au	(02) 4677 2339

Appendix A - Glossary of Terms and Definitions

Glossary of Terms and Definitions

Angle of draw	The angle of inclination from the vertical of the line connecting the goaf edge of the workings and the limit of subsidence (which is usually taken as 20 mm of subsidence).
Chain pillar	A block of coal left unmined between the longwall extraction panels.
Cover depth (H)	The depth from the surface to the top of the seam. Cover depth is normally provided as an average over the area of the panel.
Critical area	The area of extraction at which the maximum possible subsidence of one point on the surface occurs.
Curvature	The change in tilt between two adjacent sections of the tilt profile divided by the average horizontal length of those sections.
Extracted seam	The thickness of coal that is extracted. The extracted seam thickness is thickness normally given as an average over the area of the panel.
Effective extracted seam thickness (T)	The extracted seam thickness modified to account for the percentage of coal left as pillars within the panel.
Face length	The width of the coalface measured across the longwall panel.
Goaf	The void created by the extraction of the coal into which the immediate roof layers collapse.
Goaf end factor	A factor applied to reduce the predicted incremental subsidence at points lying close to the commencing or finishing ribs of a panel.
Horizontal displacement	The horizontal movement of a point on the surface of the ground as it settles above an extracted panel.
Inflection point	The point on the subsidence profile where the profile changes from a convex curvature to a concave curvature. At this point the strain changes sign and subsidence is approximately one half of S max.
Incremental subsidence	The difference between the subsidence at a point before and after a panel is mined. It is therefore the additional subsidence at a point resulting from the excavation of a panel.
Overlap adjustment factor	A factor that defines the ratio between the maximum incremental subsidence of a panel and the maximum incremental subsidence of that panel if it were the first panel in a series.
Panel	The plan area of coal extraction.
Panel length (L)	The longitudinal distance along a panel measured in the direction of (mining from the commencing rib to the finishing rib.
Panel width (Wv)	The transverse distance across a panel, usually equal to the face length plus the widths of the roadways on each side.
Panel centre line	An imaginary line drawn down the middle of the panel.
Pillar	A block of coal left unmined.
Pillar width (Wpi)	The shortest dimension of a pillar measured from the vertical edges of the coal pillar, i.e. from rib to rib.
Strain	The change in the horizontal distance between two points divided by the original horizontal distance between the points.
Sub-critical area	An area of panel smaller than the critical area.
Subsidence	The vertical movement of a point on the surface of the ground as it settles above an extracted panel.
Super-critical area	An area of panel greater than the critical area.
Tilt	The difference in subsidence between two points divided by the horizontal distance between the points.
Uplift	An increase in the level of a point relative to its original position.
Upsidence	A reduction in the expected subsidence at a point, being the difference between the predicted subsidence and the subsidence actually measured.

Appendix B – Drawings and Illustrations

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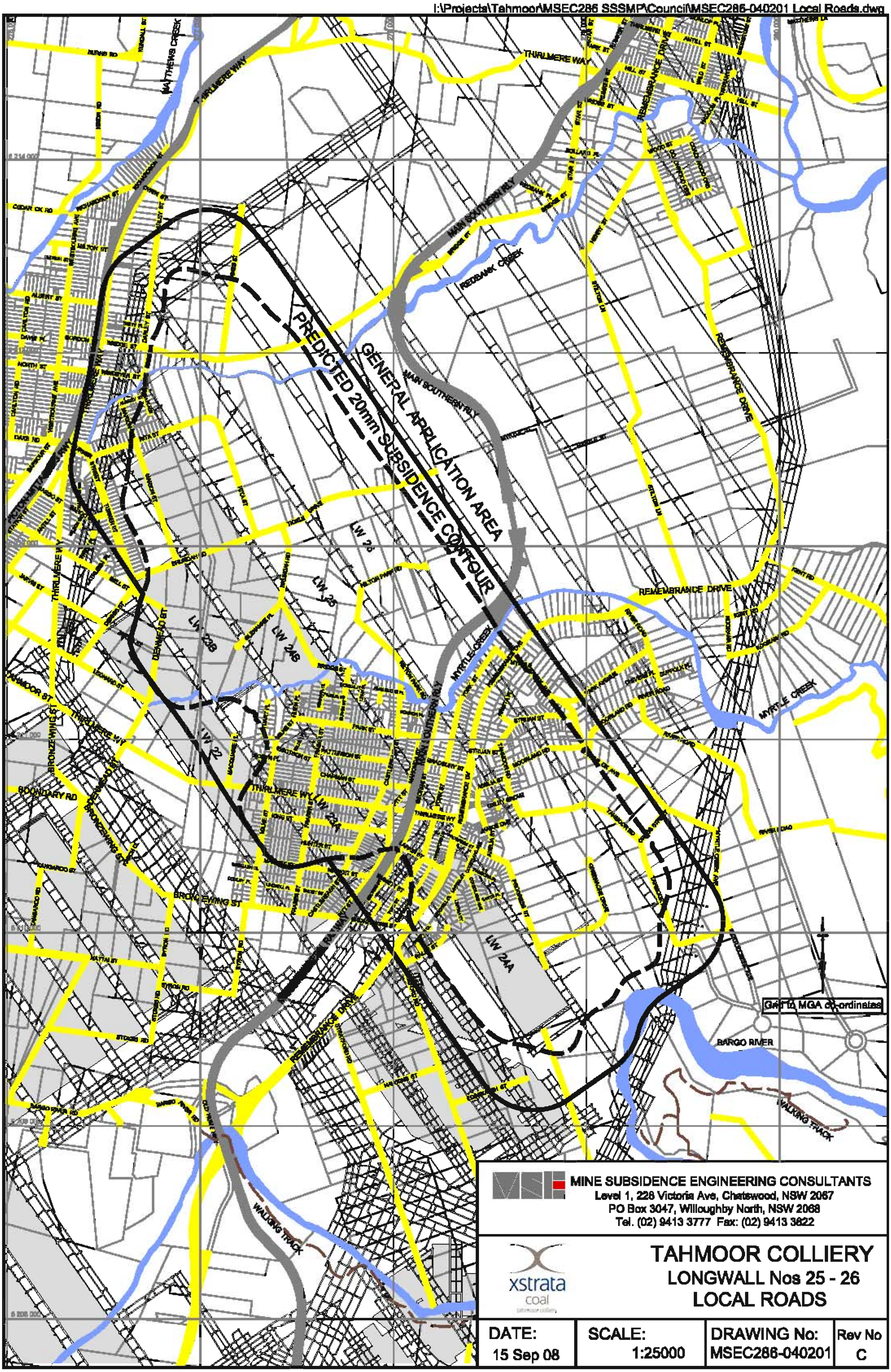

MINE SUBSIDENCE ENGINEERING CONSULTANTS
Level 1, 228 Victoria Ave, Chatswood, NSW 2067
PO Box 3047, Willoughby North, NSW 2068
Tel. (02) 9413 3777 Fax: (02) 9413 3822

xstrata coal

TAHMOOR COLLIERY
LONGWALL Nos 25 - 26
LOCAL ROADS

DATE:	SCALE:	DRAWING No:	Rev No
15 Sep 08	1:25000	MSEC286-040201	C

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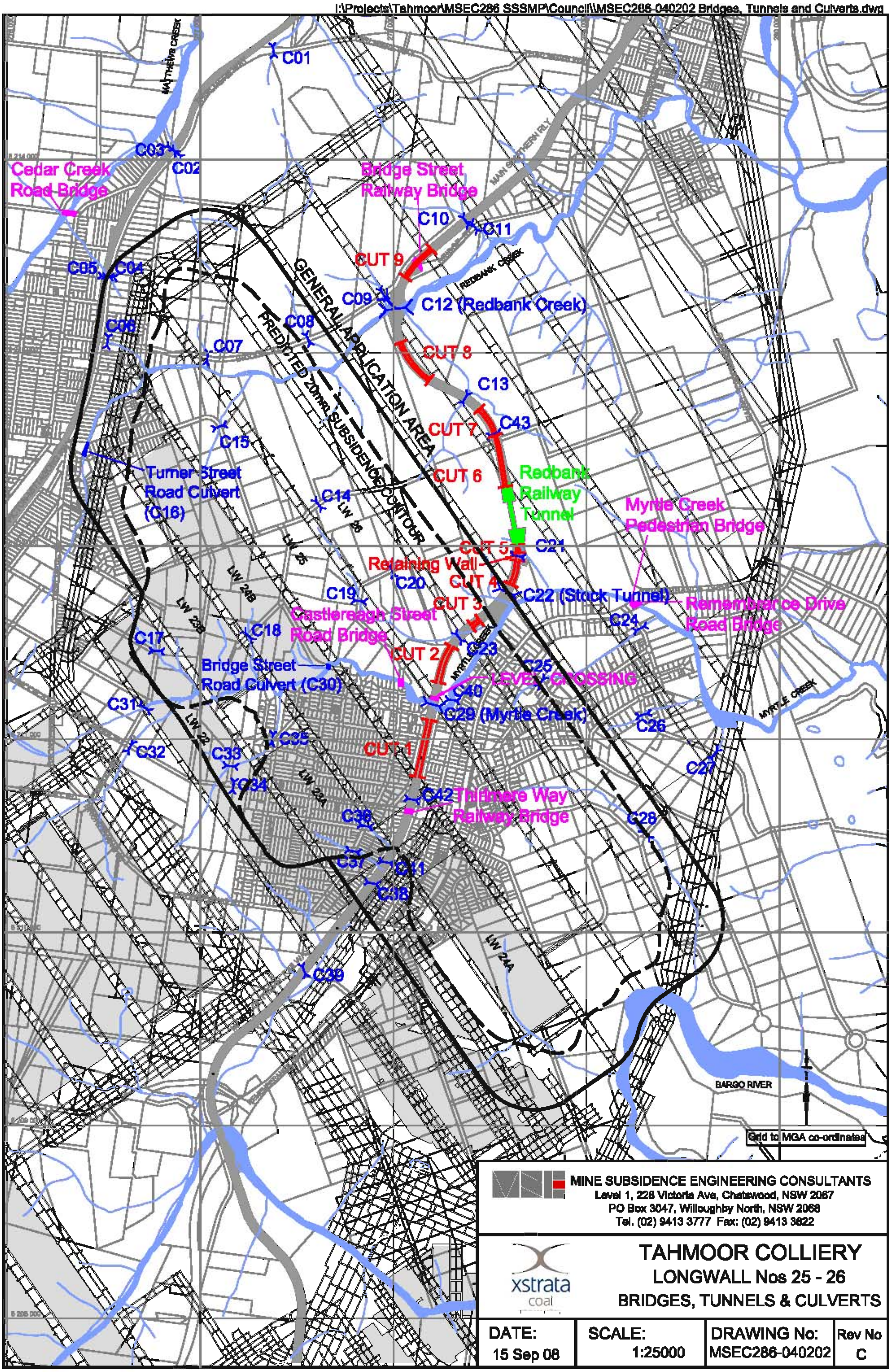

Appendix C – Subsidence Predictions for Culverts

Table D.1 - Identification and Subsidence Predictions for Culverts

Culvert	Watercourse	Feature Above	Type	Size (mm)	Location to LWs	Subsidence after LW26 (mm)	Maximum Tilt after LW26 (mm/m)	Maximum Systematic Tension during or after LWs 22-26 (mm/m)	Maximum Systematic Compression during or after LWs 22-26 (mm/m)	Upsidence after LW26 (mm/m)	Closure after LW26 (mm/m)	Compressive Strain due to Closure after LW26 (mm/m)
C04	Tributary	Road	Concrete	600	Not directly above LWs. Near end of LW25	< 20	< 0.1	< 0.1	< 0.1	< 5	< 5	0.2
C06	Redbank	Road	Concrete	900	Not directly above LWs. Near end of LW25	< 20	< 0.1	< 0.1	< 0.1	< 5	< 5	0.3
C07	Tributary	Road	Concrete	600	Above LW26	289	2.6	0.3	0.2	12	10	1.1
C08	Tributary	Road	Concrete	450	Not directly above LWs. Near side of LW26	< 20	0.1	< 0.1	< 0.1	< 5	< 5	0.5
C14	Unnamed	Road	Concrete	700	Above LW26	588	0.3	0.5	0.5	22	15	1.6
C15	Tributary	Road	Concrete	1200	Above LW25	649	2.3	0.3	0.3	39	17	1.8
C16	Unnamed	Road	Concrete	1500	Not directly above LWs. Near LW24B	< 20	< 0.1	< 0.1	< 0.1	22	8	0.9
C17	Tributary	Road	Concrete	600	Above LW22	692	1.9	0.2	0.4	42	22	2.4
C18	Tributary	Road	Concrete	750	Above LW24B	649	2.1	0.5	0.4	28	19	2.0
C19	Tributary	Road	Concrete	600	Above LW25	560	0.7	0.6	0.4	42	30	3.1
C20	Tributary	Road	Concrete	600	Above LW26	420	4.7	0.4	0.3	20	12	1.4
C25	Unnamed	Road	Concrete	450	Not directly above LWs. Near side of LW26	< 20	0.1	< 0.1	< 0.1	11	12	1.3
C28	Unnamed	Road	Concrete	600	Not directly above LWs. Near end of LW26	21	0.2	< 0.1	< 0.1	9	9	1.0
C30	Myrtle	Road	Concrete	1500	Above LW24B	647	1.4	0.5	0.3	114	88	7.4
C33	Tributary	Road	Concrete	900 x 375	Above LW22	445	2.4	0.4	0.7	34	14	1.5
C34	Tributary	Road	Concrete	900 x 375	Above LW22	525	0.8	0.4	0.9	36	13	1.4
C35	Tributary	Road	Unknown - buried	Unknown	Above LW23A	96	0.5	< 0.1	< 0.1	34	15	1.7
C36	Unnamed	Road	Concrete	380	Above LW23A	260	2.5	0.5	0.1	25	15	1.6
C37	Unnamed	Road	Concrete	450	Above LW23A	422	3.8	0.1	0.8	33	12	1.4
C40	Myrtle	Road	Concrete	600	Above LW26	623	0.6	0.5	0.5	47	32	3.3

Appendix D – Report by John Matheson & Associates on Castlereagh Street Bridge over Myrtle Creek

Castlereagh Street Bridge, Tahmoor: Trigger Report.

Prepared by John Matheson

Date: 24th November 2008.

Rev B

Our Ref: JM081112

John Matheson & Associates Pty Ltd

Consulting Civil & Structural Engineers

2/1767 Pittwater Road

Mona Vale NSW 2103

Tel: 9979 6618

Fax: 9999 0121

Email: jma.eng@bigpond.net.au

Introduction

This trigger report has been prepared by Mr. John Matheson from this office at the request of MSEC on behalf of Tahmoor Colliery and is based upon the previous reports JM061109-Rev 1 and JM070210 and the structural strengthening measures carried out during 2008 as documented on JMA drawing 0622-100.

In the event of abutment closure, it is expected that the concrete hob behind the abutment will initially crack and shear. This impact is not expected to result in any reduction in stability of the bridge. As abutment closure increases, the steel dowels that connect the deck to the abutment walls are expected to shear. This impact is not expected to result in any long-term structural concerns as the abutment walls were designed as freestanding cantilevers. However, as discussed in previous reports, we have identified a potential risk where cracking may occur to the abutment corbel at the time the dowels shear and that this cracking may occur suddenly as the abutment support transfers from a propped cantilever to a freestanding cantilever.

The purpose of the strengthening measures was to provide a lateral support for the opposing abutment walls by fastening galvanised steel brackets to the underside of the bridge deck and providing elastomeric bearings to transmit load between the abutment walls and the brackets during the event of dowel shear. The support from the brackets and bearings substantially reduces the risk of cracking of the concrete abutment if dowel shear failure were to occur due to valley closure and is intended to avert a sudden change in behaviour of the abutment wall.

Once the dowels have sheared, which is expected to have occurred after 10 mm of differential movement between the abutment and the deck, the brackets have achieved their purpose and a decision can be made by the management team as to whether to remove the brackets from at least one side of the bridge.

If subsidence monitoring indicates that little additional closure is projected to develop as mining progresses, it may be possible to leave the galvanised steel brackets in place permanently. This is because the brackets and bearings have been designed to accommodate some additional abutment closure movement beyond the point at which the dowels are expected to shear.

Elastomeric bearings were specified so that some displacement at the top of the wall could occur if the load capacity of the existing restraint elements is exceeded as a result of the inwards movement of the abutment walls due to valley closure and upsidence.

The bracket restraint system was primarily designed to allow for some movement to occur between the abutment wall and the bridge deck whilst providing a lateral load support at the top of the abutment wall assuming the lateral load bearing capacity of the existing restraint restraining structure is suddenly exhausted either through shear failure of a concrete corbel or shearing of the sleeved galvanised steel dowels (refer earlier reports for details). The elastomeric bearings were specified to allow for some inwards movement of the abutment wall under valley closure to limit the extent that passive earth pressures may develop behind the wall to minimize wall bending moments that may generate concrete wall cracking.

However, if abutment closure continues to develop, the behaviour of the elastomeric bearings is considered to be non-liner elastic up to the set point, beyond which some permanent deformation of the elastomeric bearing may occur. Typically, the compressive stress in a rubber elastomeric bearing increases in accordance with the following function, $\sigma = G \times (\lambda - \lambda^{-2})$, where λ = ((initial bearing dimension -bearing deformation)/initial bearing dimension). The load-deformation behaviour of the elastomeric bearings has been used to derive the load deformation response between the abutment wall and the bridge deck assuming the brackets behave as rigid members and the bearings are all bearing uniformly and this data is presented in figure 1.

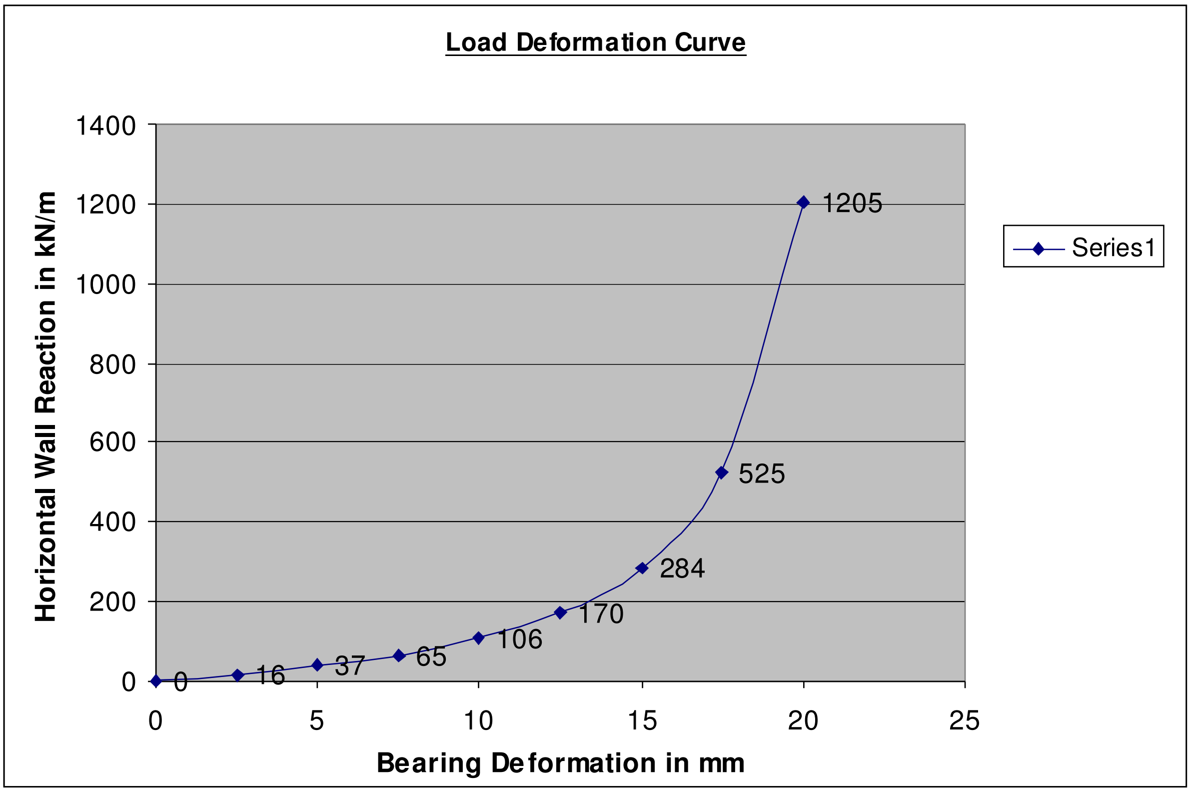

Figure 1

The principle areas of interest in relation to the bridge structure is the load/displacement behaviour of the existing and complimentary supplementary lateral support elements that provide lateral support to the top of the abutment walls and the behaviour of the abutment head and wing walls under valley closure and upsidence.

Lateral Load/Displacement Behaviour of the Abutment Supports

If abutment closure develops significantly, it is possible that the brackets, after achieving their design purpose of preventing abutment cracking at time of dowel shear, may reach the limit of their bearings and begin to apply lateral loads directly between the steel bracket and the concrete abutment walls.

A 15mm projection of the elastomeric bearing beyond the bearing keeper plates suggests that 15mm of movement can take place before the steel bears against the abutment wall and rigid support conditions arise. Reference to Figure 1 shows a horizontal wall reaction of around 284kN corresponds to a wall movement (bearing displacement) of 15mm.

The ultimate bracket capacity is calculated to be approximately 548kN at the strength limit state based upon in the HILTI method of calculation and assuming the anchor bolts have been installed as specified.

The calculated ultimate bracket capacity is approximately 1.9 times the reaction that causes 15mm bearing compression and subsequent bearing of steel on concrete although the bearing deformation would only increase 1.14 times the corresponding deformation. Once the steel bears upon the concrete wall, reactions may increase quickly in response to ongoing valley closure although 30mm of valley closure (15mm at each abutment) out of the 55mm predicted by MSEC will have occurred.

If valley upsidence occurs in conjunction with the predicted valley closure, then the abutment walls are expected to tilt outwards away from the valley centre. Whilst the base of the each abutment wall is predicted to move into the valley an outwards wall tilt will lessen the impact of valley closure at the bracket supports. An outwards wall tilt of around 2mm/m will effectively reduce the valley closure at bridge deck level as follows:

Closure at deck level =55mm – 2x6.5m x 2mm/m = 29mm, which corresponds to the 30mm closure available within the bearings.

The valley closure can be monitored at the base of the abutment walls and at the bridge deck level; wall tilt can be measured at creek bed level and the bearing deformation can be assessed visually from the ground initially and then from a scaffold to assess the performance of the bearings and the need and timing of adjustment or removal of the bearings or brackets during the active subsidence period. From these considerations, trigger levels corresponding to valley closure have been determined and are recorded n Figure 2.

It will be necessary to temporarily prop the abutments when removing the brackets, as the brackets and abutment walls will be under load. Once the brackets are removed, the jacking force in the props can be gradually reduced. It is likely that brackets will only need to be reduced on one abutment only, allowing one end to close freely.

A decision on whether to adjust or remove brackets can be made during the active subsidence period based on actual observations.

Concrete Abutment Head and Wing Wall Cracking

The causes of cracking in reinforced concrete structures are generally caused by the response of the structure to loads and/or deformations or due to the restraint of members influenced by shrinkage, temperature changes or early age plastic setting cracks.

The abutment walls already exhibit some fine cracking generally less than 0.3mm in width, which may be a response to the abutment wall loading, concrete shrinkage or temperature effects. Historically, concrete design standards have sought to limit the crack width for externally exposed structures to around 0.3mm at service loads largely for aesthetic reasons as larger cracks are likely to impair appearance and create public alarm.

At the other end of the scale, the strength limit state of the wall is of primary concern and an estimate of possible crack width at this limit state would be helpful to warn of an impending structural failure.

Reference has been made to work carried out by Park and Paulay and the CEB standards and the CEB equation for crack spacing indicates that horizontal wall cracking could be expected at around a 490mm spacing. The vertical tensile strain on the face of the abutment wall at the strength limit state is calculated to be 0.016 or 1.6%, which corresponds to a single 7.8mm crack width ignoring tension stiffening or a number of smaller closely spaced cracks.

The difficulty with specifying a cracking trigger to coincide with the strength limit state is the random nature of flexural cracking and whether or not they are identified as single more moderate cracks or part of a group of cracks that amount to impending structural failure. For this reason it has been decided that for single widely spaced cracks the yellow trigger will be set at 1.0mm and the orange trigger at 5.0mm crack widths.

Recommended Triggers

The following triggers have been established from the review of the existing reports and additional analysis carried out in the preparation of this report.

Trigger	Damage Category	Abutment Movement Relative to the Bridge Deck	Comment
	Category 0 observed crack width <0.3mm	<20mm movement between the abutment wall and the bridge deck.	Monitor weekly during active subsidence period.
	Category 1 single cracks 0.3mm to 1.0mm spaced at approx 500 centres.	20mm total movement comprising 10 .0mm movements at each abutment wall due to bearing compression.	Monitor weekly. Install crack monitoring gauges on 1.0mm cracks and measure rate of crack growth. Structural inspection of the abutment wall. Consider installing temporary horizontal props between the abutment walls to adjust or remove the galvanised steel brackets.
	Category 2 single cracks 1.0mm to 5.0mm spaced at approx 500mm centres	30mm total movement comprising 15.0mm movement at each abutment wall due to bearing compression.	Monitor daily. Monitor rate of abutment wall crack growth. Consider installing horizontal props to strut the abutment walls to reduce the impact of earth pressure on the reinforced concrete wall if brackets have not been removed. Weekly structural inspection of abutment wall cracking required.
	Category 3 single cracks in	60mm total movement comprising 30mm movement at each	The abutment wall and may be nearing the strength limit state and the restraint brackets are nearing

	excess of 5.0mm spaced at approx 500mm centres.	abutment due to 15mm bearing compression plus 15mm bracket adjustment.	bearing closure if they have not been removed. Consider limiting bridge traffic.

Figure 2

Appendix E – Management Plan for Thirlmere Way Overbridge

Tahmoor Colliery
Longwalls 25 to 26

THIRLMERE WAY RAILWAY OVERBRIDGE

SURFACE SAFETY AND SERVICEABILITY MANAGEMENT PLAN

REVISION D

Prepared by Technical Committee

October 2008

REVIEW

Date	Rev	Comments
14-Jun-06	A	Draft for Submission to ARTC
27-Jun-06	B	Revised by GHD
20-Sep-07	C	Updated for LW24A (bridge works complete and contacts updated)
15-Sep-08	D	Updated for LW25

REFERENCES

AS/NZS 4360:1999 Risk Management

GHD, (2008). *Surface Safety and Serviceability Management Plan for Longwall Mining under the Main Southern Railway*. ARTC Tahmoor Colliery Technical Committee, October 2008.

JMA (2005a). *Structural Investigation Report for Picton to Mittagong Deviation Overbridge at Tahmoor subject to Prescribed Ground Movements Caused by Predicted Mine Subsidence.* John Matheson and Associates, 2005.

JMA (2006a). *Design for Strengthening Works for Thirlmere Way Overbridge* John Matheson and Associates, Revision C, 2006.

Drawing Number	Drawing Title	Amendment
GN01	General Notes Sheet 1	A
GN02	General Notes Sheet 2	E
GN03	Design Loading Diagrams	B
GN04	On Site Work Sequence and Traffic Impact Vehicle Load and Speed Limits	E
1.00	Existing Bridge Plan and Details	C
1.01	New Bridge Plan and Details	G
1.02	Abutment and Wing Wall Repair and Alterations	C
2.00	Abutment Strengthening Plan	G
2.01	Stage 1 Restraining Plate Plan and Details	D
2.02	Abutment Strengthening Details	G
3.00	Capping Beam Plan	G
3.01	Capping Beam Details and Ground Anchor Details	F
4.00	Plan Layout of Deck Plates	D
4.01	New Bridge Deck Plan	D
4.02	Bridge Deck Details Sheet 1	D
4.03	Bridge Deck Details Sheet 2	B
5.00	Road Pavement Plan	B
5.01	Road Pavement Details	B

JMA (2008). *Thirlmere Way Overbridge Bearing Triggers, Tahmoor*. John Matheson and Associates, 2008.

MSEC, (2006a). *Tahmoor Colliery Longwalls 24 to 26 - The Prediction of Subsidence Parameters and the Assessment of Mine Subsidence Impacts on Surface and Sub-Surface Features due to mining Longwalls 24 To 26 at Tahmoor Colliery in support of an SMP Application*. Report No. MSEC157, Revision B, Mine Subsidence Engineering Consultants, March 2006.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

1.1. Background

Tahmoor Colliery proposes to extract coal from the Bulli Seam directly beneath the Main Southern Railway using longwall mining techniques. No impacts were observed at the Thirlmere Way Overbridge during the mining of Longwalls 22 to 24. The Thirlmere Way Overbridge is located directly near the commencing end of Longwall 24B and is expected to experience additional mine subsidence movements during the mining of Longwalls 25 to 26.

Fig. 1.1 Thirlmere Way Railway Overbridge

The original two-lane bridge was constructed with masonry abutments and a single-span concrete deck. The bridge was designed and constructed in 1919.

As part of developing a subsidence management plan for Longwalls 24 to 26, Tahmoor Colliery conducted two studies into the potential impacts of mine subsidence on the Thirlmere Way Railway Overbridge. These studies included predictions of mine subsidence parameters (MSEC, 2006a) structural investigations and impact assessments (JMA, 2005a), and structural design of strengthening works for the Overbridge (JMA, 2006a). The strengthening works were completed prior to the commencement of Longwall 24B.

This Management Plan provides detailed information about how the risks associated with mining near the Thirlmere Way Overbridge will be managed by Tahmoor Colliery and ARTC. The Management Plan is a live document that can be amended at any stage of mining, to meet the changing needs of Tahmoor Colliery and ARTC.

1.2. Location

Thirlmere Way Railway Overbridge provides direct access between the Tahmoor retail and commercial areas and the Tahmoor urban area located west of the Main Southern Railway. A plan showing the location of the Overbridge is shown in Fig. 1.2 and a photograph is shown in Fig. 1.1.

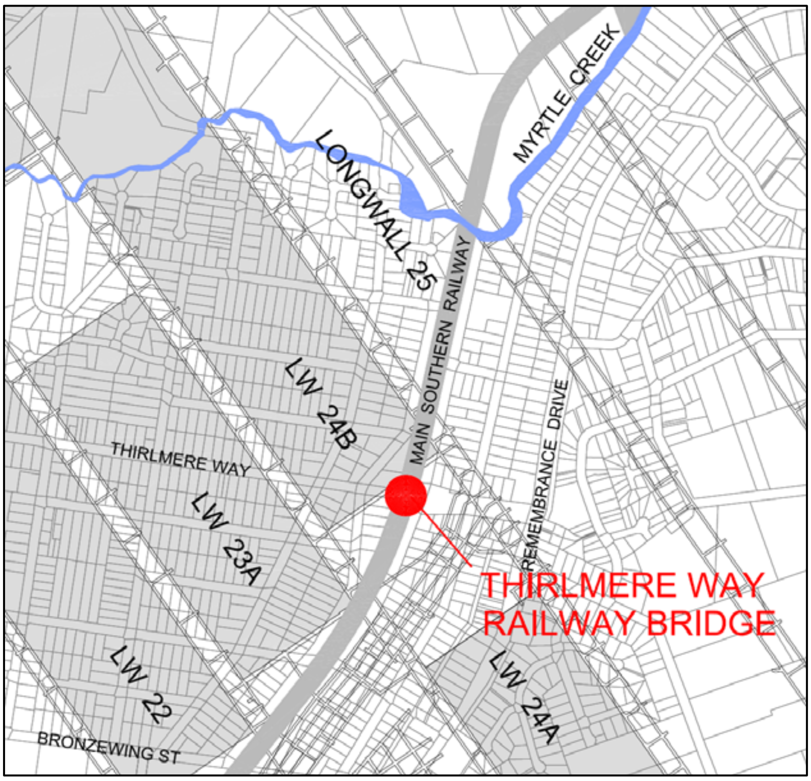

Fig. 1.2 Location of Thirlmere Way Overbridge

The Thirlmere Way Overbridge is located at Railway chainage 94.300 km. It is located approximately 140 metres from the closest edge of Longwall 25.

1.3. Relationship to Other Documents

This Management Plan forms part of an overall Management Plan for the Main Southern Railway. The plan should be read in conjunction with this overall Management Plan.

1.4. Purpose of the Plan

The purpose of this Management Plan is to describe in detail the methodology which will be utilised to manage the risks associated with the Thirlmere Way Overbridge during mining.

The plan is also designed to illustrate that Tahmoor Colliery, through the Steering Committee, Technical Committee and Sub-Committees has the technical knowledge and capacity to manage the effects of mine subsidence on the Overbridge and is able to respond to any outcomes that could potentially affect the operation of the Main Southern Railway.

1.5. Objectives

The objectives of this Management Plan are to establish procedures to measure, control, mitigate, repair and rehabilitate potential impacts that might occur on the Thirlmere Way Overbridge as a result of longwall mining. The objectives have been developed to:-

- Ensure the safe and serviceable operation of the Main Southern Railway and Thirlmere Way. Public and workplace safety is paramount. Disruption and inconvenience to travelling public is to be minimised.
- Avoidance of disruption and inconvenience to ARTC operations.
- Initiate action to mitigate or remedy potential significant impacts that might occur.
- Provide a plan of action in the event that the impacts develop.
- Provide a forum to report, discuss and record impacts for all relevant parties.
- Establish lines of communication and emergency contacts.

1.6. Scope

The Management Plan is to be used to protect and monitor the condition of the Thirlmere Way Overbridge. The items at risk are:-

- Bridge deck
- Associated wingwalls and embankments

The Management Plan only covers infrastructure associated with Thirlmere Way Overbridge and its strengthening works. The management plan does not include the railway track, for which a separate management plan has been developed (GHD, 2008).

CHAPTER 2. RISK MANAGEMENT METHOD

2.1. General

The Australian/New Zealand standard for Risk Management defines the terms used in the risk management process, which includes the identification, analysis, assessment, treatment and monitoring of risk. In this context:-

2.1.1. Consequence

'The outcome of an event expressed qualitatively or quantitatively, being a loss, injury, disadvantage or gain. There may be a range of possible outcomes associated with an event.'[1] The consequences of a hazard are rated from very slight to very severe.

2.1.2. Likelihood

'Used as a qualitative description of probability or frequency.'[2] The likelihood can range from very rare to almost certain.

2.1.3. Hazard

'A source of potential harm or a situation with a potential to cause loss.'[3]

2.1.4. Risk

'The chance of something happening that will have an impact upon objectives. It is measured in terms of consequences and likelihood.'[4] The risk combines the likelihood of an impact occurring with the consequence of the impact occurring. The risk is rated from very low to extreme. In this study, the likelihood and consequence are combined via the qualitative risk analysis matrix shown in Table 2.1, to determine an estimated level of risk for particular events or situations.

The Risk Analysis Matrix is similar to the example provided in AS/NZS 4360:1995, Appendix D, p.25.

Table 2.1 Qualitative Risk Analysis Matrix

LIKELIHOOD	CONSEQUENCES				
	Very Slight	**Slight**	**Moderate**	**Severe**	**Very Severe**
Almost Certain	Low	Moderate	High	Extreme	Extreme
Likely	Low	Moderate	High	Very High	Extreme
Moderate	Low	Low	Moderate	High	Very High
Unlikely	Very Low	Low	Moderate	High	High
Rare	Very Low	Very Low	Low	Moderate	High
Very Rare	Very Low	Very Low	Low	Moderate	Moderate

This SSSMP adopts a common system of nomenclature to summarise each risk analysis, which is **"LIKELIHOOD / CONSEQUENCE → LEVEL OF RISK".**

For example, if the likelihood of a risk is assessed as "**UNLIKELY**", and the consequence of a risk is assessed as "**SEVERE**", the risk analysis would be summarised as **"UNLIKELY / SEVERE → HIGH"**.

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[1] AS/NZS 4360:1999 – Risk Management pp2
[2] AS/NZS 4360:1999 – Risk Management pp2
[3] AS/NZS 4360:1999 – Risk Management pp2
[4] AS/NZS 4360:1999 – Risk Management pp3

CHAPTER 3. INVESTIGATIONS AND STUDIES

3.1. Observed Movements at the Thirlmere Way Overbridge

Surveys of subsidence, tilt and strain were last conducted on the Thirlmere Way Overbridge on 30 July 2008. The Bridge has subsided a total of 98 mm since the bridge strengthening works were completed.

Measured tilts and strain at the abutments are very small. The results indicate a fall of up to 1.0 mm/m between the bridge abutments from east to west, towards Longwall 24B.

Measured differential movements between the abutments and across the expansion joint are also very small, as shown in Fig. 3.1. It is noted that the cover plates over the expansion joint on the bridge deck were adjusted to resolve noise issues (not due to mine subsidence movements).

No impacts have been observed at the Bridge during the mining of Longwalls 24B and 24A.

3.2. Subsidence Predictions for Longwalls 25 and 26

Predictions of subsidence, tilt and strain at the centre of the Bridge were provided in a report by MSEC (2006a). It is predicted that the bridge will experience an additional 70 mm of subsidence during the mining of Longwalls 25 and 26.

It is noted, however, that observed subsidence (98 mm)was greater than predicted subsidence (82 mm) during the mining of Longwalls 24B and 24A. However, the difference in subsidence is very small.

There is a barrier of coal left between Longwalls 22 to 24B (on the northern side) and the 200 Panels and Longwall 24A (on the southern side). This coal has not been extracted, except for development headings.

Longwall 25 will excavate coal directly across this barrier, effectively blocking "the corridor". The barrier of coal is located directly beneath the Main Southern Railway and the path of the Railway runs diagonally across Longwall 25 from the corner of the barrier.

The mining geometry in the vicinity of the coal barrier is rare in the Southern Coalfield. A similar mining geometry was present between Tower Longwalls 6 to 14, but unfortunately, limited subsidence monitoring was undertaken, most of which was influenced heavily by the presence of the nearby deeply incised Cataract Gorge. However, there is some empirical data measured around and directly above other coal barriers that provide some indication of possible deviations from the predictive model.

It is also possible that increased vertical subsidence will be observed along the railway where it crosses directly above the coal barrier. There have been a number of examples, including locations above Tahmoor Colliery, where subsidence monitoring has shown increased vertical subsidence of the surface in areas that are located directly above an isolated coal barrier. The magnitude of settlement has typically been between 50 and 150 mm above what would be predicted using the Standard Incremental Profile Method. The cause of the additional subsidence has not been proven, but it is thought that it is a result of a general relaxation of in-situ stresses in the strata within the coal barrier.

While observed subsidence may exceed predictions for the section of railway above the coal barrier, subsidence monitoring has shown that it is usually accompanied by relatively low systematic tilts, curvature and strains. This is discussed in further detail in Section 3.2.1.

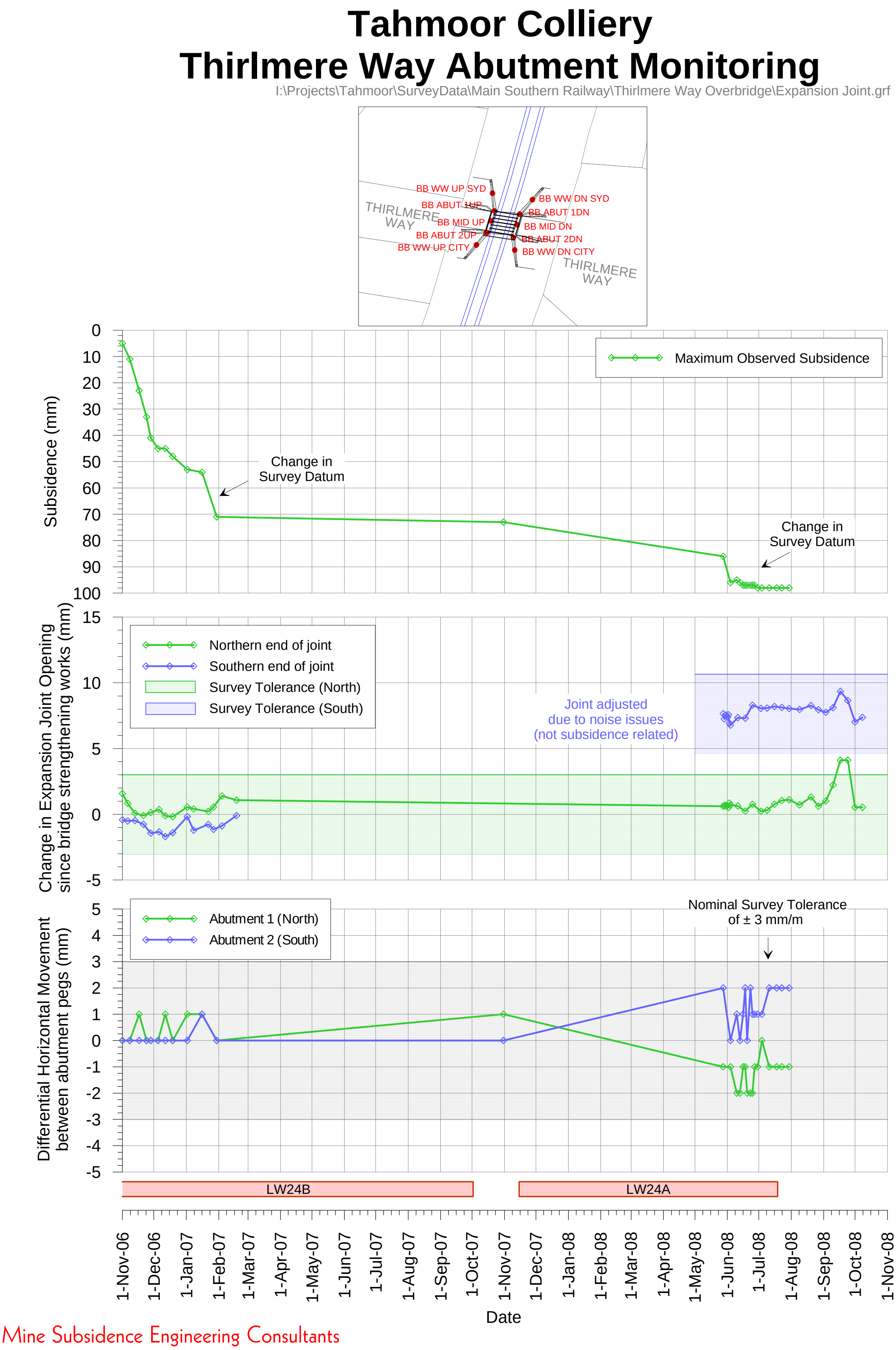

Fig. 3.1 Observed differential horizontal movements between abutments

3.2.1. Frequency Analysis of Ground Strains beyond goaf edge at Tahmoor Colliery

Given that the Overbridge is 140 metres from Longwall 25 and above solid coal, an analysis was conducted of observed strains measured between survey pegs that were between 0 and 200 metres from the side of all previous active longwalls, where pegs were located above solid coal and had not been directly extracted beneath. The dataset included observed strains between survey pegs from goaf edge (0 metres) rather than 100 metres as a conservative measure.

A plan showing all of the survey pegs included within the analysis is provided in Fig. 3.2. In each case, the survey pegs were only used when the active longwall was between 0 and 200 metres from the nearest side, and only when the pegs were located over solid coal.

Fig. 3.2 Locations of Survey Pegs used for Frequency Analysis of Observed "Solid Coal" Ground Strains at Tahmoor Colliery

A frequency analysis was conducted to estimate the frequency of observed "solid coal" ground strains at Tahmoor Colliery that were measured during the mining of each previous longwall. If multiple surveys were undertaken during the mining of a longwall, the maximum tension or compression was recorded and used for the analysis. The results of frequency analysis are shown graphically in Fig. 3.3.

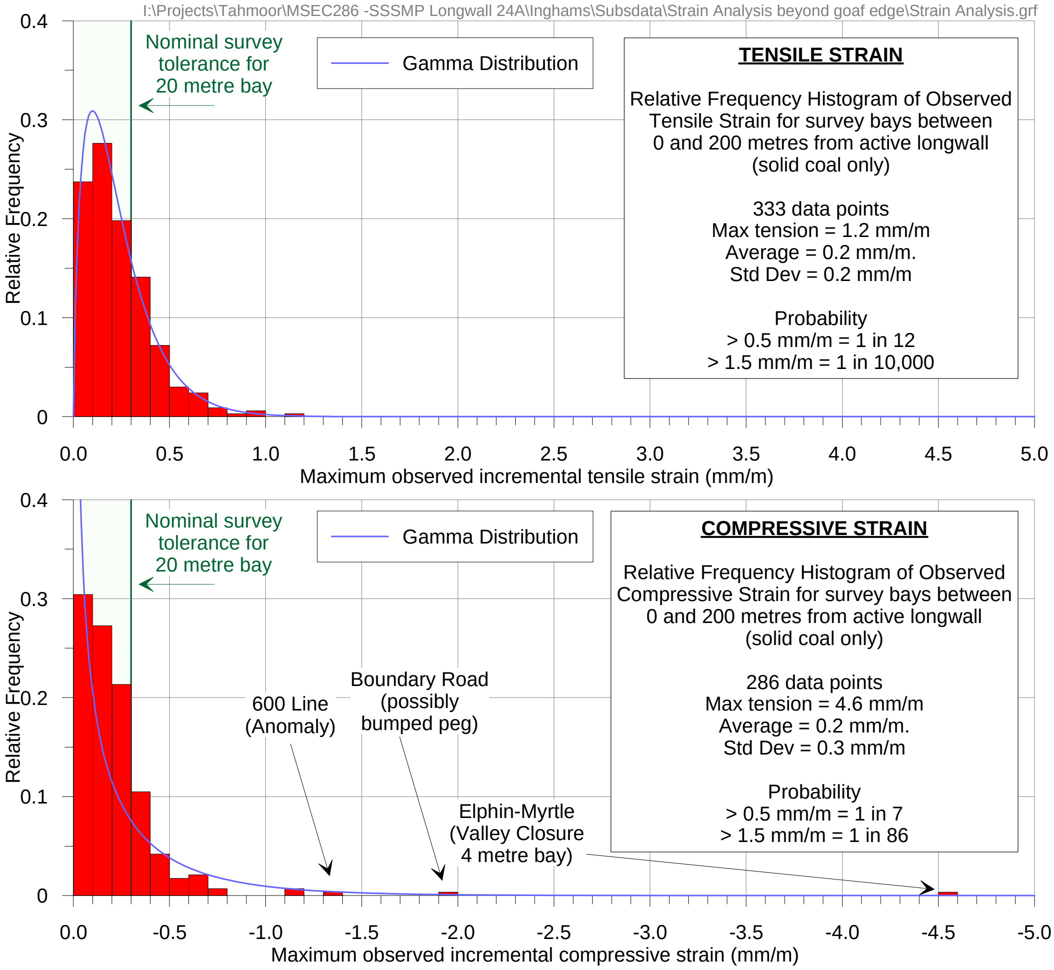

Fig. 3.3 Results of Frequency Analysis of Observed "Solid Coal" Strains at Tahmoor Colliery

It can be seen from the results that the average observed tensile or compressive "solid coal" strains at Tahmoor Colliery for this dataset is 0.2 mm/m, which are within survey tolerance. With respect to observed tensile strains, very few ground strains have exceeded 0.5 mm/m and none have exceeded 1.2 mm/m. Based on a conservatively selected dataset, the probability that tensile strains will exceed 0.5 mm/m is 1 in 12 and the probability that tensile strains will exceed 1.5 mm/m is 1 in 10,000 based on nominal 20 metre bays.

With respect to observed compressive strains, some strains have been observed that are greater than 1.5 mm/m. The maximum observed strain of 4.6 mm/m was measured during the mining of Longwall 24B in the base of Myrtle Creek, where such movements are expected. The rest of the observed ground strains are less than 2.0 mm/m, where one location is considered to be possibly survey error but has been included in the analysis as a conservative measure. Based on a conservatively selected dataset, the probability that compressive strains will exceed 0.5 mm/m is 1 in 7 and the probability that compressive strains will exceed 1.5 mm/m is 1 in 86 based on nominal 20 metre bays.

3.3. Structural Investigations and Impact Assessment

3.3.1. Initial Investigations and Assessments

The pre-mining condition and structural stability of the bridge had been assessed by John Matheson & Associates (2005a). As discussed in the report, the bridge did not conform to current Australian Standards in its original state and some evidence of stress was apparent in the bridge. The report recommended that the bridge should be strengthened and that the bridge could accommodate mine subsidence movements if strengthening works were undertaken.

3.3.2. Bridge Strengthening Works

Tahmoor Colliery engaged John Matheson & Associates to provide structural designs for strengthening the Overbridge, which commenced in early 2006 (JMA, 2006a).

As the design of the bridge progressed, it became apparent that it would be more cost-effective to replace the bridge deck and parapet wall, rather than modify the connection between the abutment walls and the bridge deck. The design of the strengthening works included the following components.

- Drilling, installing and grouting passive steel reinforcement to the brick abutments.
- Drilling, installing and grouting raking post-tensioned ground anchors.
- Replacement of the existing bridge deck with a new galvanised structural steel bridge deck.

The design of the strengthening works allow the Overbridge to accommodate the predicted mine subsidence movements, with appropriate factors of safety, in the following ways.

- The abutment walls can now support the active earth pressures and vehicle loads by themselves. The original abutments relied on the bridge deck for support.
- The abutment walls have been strengthened with steel reinforcement, which provides for ductile rather than brittle modes of failure.
- The new bridge deck is fixed to one abutment and allowed to slide above the other abutment. This will allow the bridge to accommodate ground strains associated with mine subsidence. The original bridge was fixed at both abutments.

These designs were independently reviewed and certified by Cardno MBK Pty Ltd. Approval for Tahmoor Colliery to undertake the strengthening works was provided by ARTC and RIC.

The bridge strengthening works commenced in May 2006 and are now complete. The existing bridge deck was replaced in June 2006.

Fig. 3.4 Thirlmere Way Railway Overbridge

CHAPTER 4. Risk Assessment

4.1.1. Structural Damage to the Overbridge

Whilst the bridge strengthening works have greatly improved the ability of the bridge to withstand mine subsidence movements, there remains a risk that the bridge will experience impacts as a result of mine subsidence movements. These impacts include cracking to the abutment walls and, in the most extreme case, failure of the bridge structure.

Mine subsidence movements may impact the bridge in a number of ways.

- *Ground strain orientated in a direction longitudinal to the bridge*

  Additional structural advice on the ability of the Bridge to withstand abutment opening and closure has been provided by John Matheson & Associates (2008).

  JMA advises that abutment closure does not pose an immediate structural concern to the bridge. If the abutments close, the deck will slide in response and if the closure exceeds 100 mm, it is expected that the concrete capping beam behind the deck will crack, which is not significant.

  The design of the bridge strengthening works provides a 130 mm clearance between the face of the abutment wall and the edge of the elastomeric bearing supporting the bridge deck. The limit of abutment opening before the bridge bearing comes into contact with the edge of the corbel is 130mm. This equates to a ground strain of approximately 14 mm/m. As discussed in Section 3.2.1, the probability of tensile strains exceeding 14 mm/m at the Bridge is substantially smaller than 1 in 10,000.

  JMA have also considered the unlikely scenario that the pre-greased slip joint between the deck and the western abutment is partially or completely ineffective. JMA advised a yellow trigger if the abutments open by more than 20 mm if the joint is ineffective. This equates to a ground strain of approximately 2.2 mm/m. As discussed in Section 3.2.1, the probability of tensile strains exceeding 2.2 mm/m at the Bridge is smaller than 1 in 10,000. The overall probability of impact is substantially less than 1 in 10,000 once the probability that the pre-greased slip joint is ineffective is taken into account.

  The probability is considered lower than suggested by the frequency analysis given that observed differential movements across the abutments during the mining of Longwalls 24B and 24A have been 2 mm or less, which is within survey tolerance.

  The likelihood of the strengthened bridge experiencing impacts associated with longitudinal ground strain is **VERY RARE**.

- *Ground strains and curvature along the reinforced masonry abutments and wing walls*

  The masonry walls have been strengthened to accommodate ground strains along the abutments and wing walls. Based on the discussion above, the likelihood that the abutments would become unstable is assessed as **VERY RARE.**

- *Ground tilt*

  It is predicted that the bridge will experience maximum tilts of 1.3 mm/m. The Bridge has experienced tilts of 1.0 mm/m during the mining of Longwalls 24B and 24A. It is expected that the mining of Longwall 25 will reverse the direction of tilt of the Bridge.

  Ground tilt is generally considered to affect the stability of the structure only if it is greater than 10 mm/m. JMA (2008) advised that the tops of the abutment walls will open beyond 130 mm if both abutment walls tilt outwards by 10 mm/m.

  The likelihood of structural failure as a result of ground tilt is therefore assessed as **VERY RARE**.

- *Twist*

  Predictions of subsidence have been made along the abutment walls and an analysis of these results suggest that the angular change will be less than 0.001 radians, which is negligible.

  The likelihood of structural failure as a result of twist is therefore assessed as **VERY RARE**.

As shown above, it is assessed that the likelihood of structural failure occurring as a result of mine subsidence from mining Longwalls 25 to 26 is assessed to be **VERY RARE**.

If the bridge fails, it is likely that the mode of failure will be ductile rather than brittle, given the introduction of steel reinforcement to the abutment walls. However, as a conservative measure, the consequence of structural failure to the bridge is assessed as **VERY SEVERE**.

The level of risk associated with structural failure is therefore assessed as **VERY RARE / VERY SEVERE → MODERATE**.

4.1.2. Damage to the Road Pavement

The design of the bridge strengthening works provides for expansion joints in the pavement. These joints will provide for the development of ground strain in the pavement.

The likelihood of impacts to the road pavement on the bridge is therefore assessed as **VERY RARE**.

In the event of cracking to the road pavement, any damage can be easily repaired. The consequence of damage to the pavement is therefore assessed as **SLIGHT**.

The level of risk associated with damage to the road pavement is therefore assessed as **VERY RARE / SLIGHT → VERY LOW**.

4.1.3. Encroachment of Bridge Abutments and Deck inside allowable KE+200mm

The abutment walls and bridge deck currently lie outside the calculated kinematic envelope + 200mm safety margin (KE+200mm) for narrow non electric trains by a distance of approximately 465 mm vertically and 210 mm horizontally.

The predicted maximum compressive strains are 0.2 mm/m. This equates to a closure of the abutment walls of less than 2mm across the 9 metre span bridge. The predicted maximum closure of the abutments is therefore an order of magnitude less than what is required for the walls to encroach within the allowable kinematic envelope.

The likelihood of impacts to the road pavement on the bridge is therefore assessed as **VERY RARE**.

The consequence of this risk is assessed as **MODERATE**

The level of risk associated with damage to the road pavement is therefore assessed as **VERY RARE / MODERATE →LOW**.

4.1.4. Impacts to the Water Main

The Thirlmere Way Railway Overbridge supports a water main, which is owned and maintained by Sydney Water. The section of pipe supported by the Overbridge was replaced by Tahmoor Colliery. The new DICL pipe connects to the existing Sydney Water pipes with spigot and socket joints, which are similar to most other water pipe joints within the SMP area.

The risks associated with this water main have been addressed in a separate SSSMP, which has been agreed between Tahmoor Colliery and Sydney Water and have not been considered further in this SSSMP.

CHAPTER 5. RISK CONTROL PROCEDURES

5.1. Management Structure

5.1.1. Rail Maintenance Contractor Site Safety Manager

Tahmoor Colliery has appointed a Rail Maintenance Contractor to act as the site safety manager for the sections of track that will be affected by mine subsidence during the mining of each longwall.

The Rail Maintenance Contractor is responsible for:

- Assessing and certifying the track at full or restricted track speed;
- Coordinating all responses to issues that occur on site, whether they are mining or non-mining related, including reporting and closing out of all alarms;
- Ensuring that all site work is undertaken safely in the accordance with all relevant OH&S legislation by its employees and all other contractors working for Tahmoor Colliery on site;
- Undertaking all track-related work, including (but not limited to) cutting and restressing rails, tamping and adjusting ballast; and
- Direct point of contact to ARTC.

5.1.2. On-site Track Certifier

As part of the Rail Maintenance Contractor's obligations, it will provide a qualified Track Certifier on site. The Track Certifier is responsible for:

- Visual inspections of track and all structures within the rail corridor, including cuttings and embankments, right-of-way, Myrtle Creek Culvert, Tahmoor Station platform and Thirlmere Way Overbridge;
- Assessing and certifying the track at full or restricted track speed;
- Record track geometry results via "Amber" track geometry device;
- Undertake manual track geometry measurements using standard ARTC methods, if required;
- Undertake measurement of settlement plates in the baulk;
- Immediate track inspection upon notification of alarms;
- Direct point of contact with ARTC Track Manager (Moss Vale) and Train Control at Junee, and Tahmoor Colliery Control Centre; and
- Change of shift advice to ARTC Train Control.

Any actions to implement speed restrictions or stop trains will be undertaken by the Track Certifier. All communications with ARTC Train Control will be conducted via the Track Certifier. While ARTC may directly implement speed restrictions or stop trains, the Track Certifier also has authority to take these actions.

The Track Certifier will implement an action (such as a corrective action, or a speed restriction or stop trains) based on the following:

- Observations of a noticeable track exceedent;
- Advice from the Emergency Response Group, based on monitoring results and analysis; or
- Request from ARTC.

The Rail Maintenance Contractor will coordinate a team of inspectors to provide an on site Track Certifier at critical times during the mining period. The presence of a Track Certifier will vary during the mining period. Decisions to increase or reduce on site presence will be based on recommendations from the Rail Management Group and approved by ARTC. The following is planned:

- Daily inspection once switches are installed. Inspections and monitoring increase in frequency as the longwall approaches the railway at a rate depending on the rate of longwall advance and observed subsidence movements;
- Continuous on-site presence by a team of inspectors during most active subsidence. Based on subsidence predictions and experiences of mining Longwalls 22 to 24, this will occur once the longwall has approached to within 100 metres of the railway;

- Gradual reduction in on-site presence from 24/7 to daily depending on the rate of longwall advance and observed subsidence movements; and
- Once it is established that subsidence has effectively abated, daily inspection until switches are removed or the Rail Management Group and ARTC agree that a reduction in inspection frequency is satisfactory.

5.1.3. Tahmoor Colliery Control Centre

The Tahmoor Colliery Control Centre (TCCC) will make sure the Track Certifier has received notification of alarm and are acting on it. The list of contacts will vary, depending on the nature, type and severity of the alarm. If the TCCC cannot contact the Track Certifier, the operator will follow a pre-arranged contact list of back-up representatives.

5.1.4. Rail Management Group (RMG)

The RMG is responsible for taking the necessary actions required to manage the risks that are identified from monitoring of the rail infrastructure. Members of the RMG include:

- ARTC (Ross Barber);
- BMT WBM (Rod Sweeting);
- David Christie;
- DPI (Gang Li);
- GHD (Graeme Robinson);
- GHD Geotechnics (Andrew Leventhal);
- JMA (John Matheson);
- Martinus Rail (Treaven Martinus);
- Meadows Consulting (John Rolles);
- MSB (Darren Bullock);
- MSEC (Daryl Kay);
- PCE (Allan Pidgeon);
- TRT (Mark Wroblewski);
- SSS (Ted Johansen);
- On-site Track Certifier; and
- Tahmoor Colliery (Ian Sheppard).

The RMG will meet in person or via teleconference at regular intervals once the switches have been installed. As a minimum, the RMG will meet on a weekly basis during the active subsidence period. The RMG will review the monitoring results and consider whether any additional actions are required.

5.1.5. Emergency Response Group (ERG)

Some members of the RMG are required to respond immediately to alarms that are triggered by monitoring results. These members form a group called the Emergency Response Group and are "on call" at all times during the monitoring period. The members of the ERG in relation to the Bridge are shown on the list below:

- ARTC;
- GHD;
- JMA;
- MSEC;
- Rail Maintenance Contractor
- On-site Track Certifier; and
- Tahmoor Colliery.

Notification of alarms for Bridge monitoring will be performed manually by phone or email.

5.1.6. Alternative Contacts

All members of the RMG (and therefore ERG) have provided alternative contacts during the mining period. The alternative contacts can be contacted should the primary contact be unavailable. In some instances, more than one alternative is provided by RMG members.

In the case of the on-site Track Certifier, a designated mobile phone is carried by the person that is currently on call.

5.2. Mitigation measures

Substantial mitigation measures have been undertaken at the Overbridge. The design of the strengthening works allow the Overbridge to accommodate the predicted mine subsidence movements, with appropriate factors of safety, in the following ways.

- The abutment walls can now support the active earth pressures and vehicle loads by themselves. The original abutments relied on the bridge deck for support.
- The abutment walls have been strengthened with steel reinforcement, which provides for ductile rather than brittle modes of failure.
- The new bridge deck is fixed to one abutment and allowed to slide above the other abutment. This will allow the bridge to accommodate ground strains associated with mine subsidence. The original bridge was fixed at both abutments.

5.3. Monitoring Plan

The following monitoring will be undertaken in the vicinity of the Thirlmere Way Overbridge:

- Weekly 2D structure monitoring of Thirlmere Way Overbridge abutments and wingwalls, as shown in Fig. 5.1.
- Weekly 2D and monthly 3D monitoring line along the Main Southern Railway Corridor on the up side of the track, with pegs spaced nominally 20 metres apart.
- Weekly 2D monitoring along Thirlmere Way across the Bridge.
- Weekly measurement of gap in expansion joint in the bridge deck on the road pavement
- Daily visual inspections by the Track Certifier during mining. Detailed inspections will be undertaken by Sunrise Building and Property Services on a weekly basis during mining.

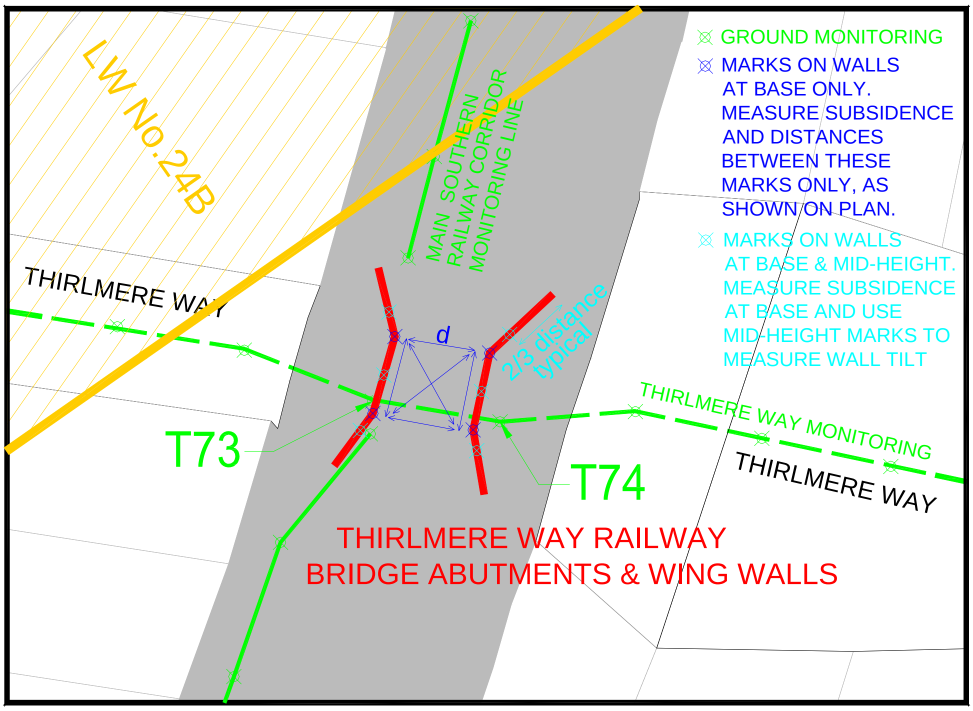

Fig. 5.1 Survey Marks on or near the Thirlmere Way Railway Overbridge

Minimum monitoring requirements based on longwall position have been developed in the risk control procedures, which will be followed regardless of the monitoring results. Minimum monitoring requirements for Longwall 25 are shown in Figure 5.1. A similar approach will be adopted for Longwall 26.

Monitoring frequencies can be increased earlier than the minimum requirements based on reviews of observed monitoring data by the RMG.

Monitoring frequencies will not be reduced or stopped until agreed by ARTC (via recommendations by the RMG) and DPI. This applies to all monitoring activities.

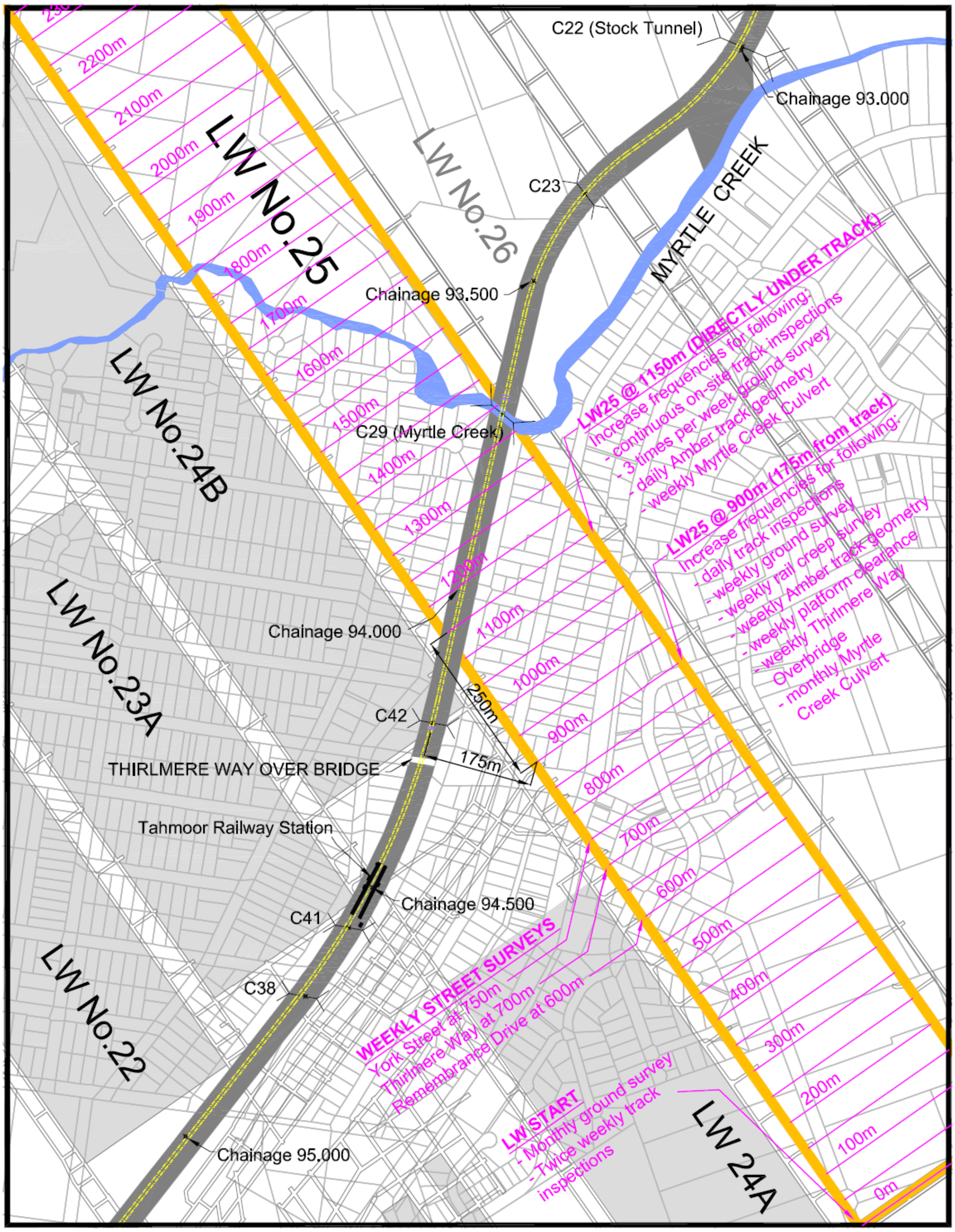

Figure 5.1 Minimum Monitoring Requirements for Surveys and Inspections

5.4. Trigger Levels

Trigger levels have been divided into four categories, which relate to the safe operation of the trains.

Table 3.1 Trigger Levels

Trigger Level	Description
GREEN	Observations within predictions. Operate as normal.
BLUE	Observations outside predictions but within operating tolerance. Investigate cause. Some action may occur to prevent operating restrictions.
YELLOW	Restrictions on operations. Action required within 6 hours. Appropriate speed restriction may apply until altered to Green or Blue Level.
RED	Stop trains, inspect prior to next train, repair to lower category, pilot trains if safe.

The YELLOW and RED triggers are directly related to the safe operation of the trains and are linked to NSW rail safety standards. The categories were first adopted for management of potential mine subsidence impacts on a railway by the Glennies Creek / Mt Owen Rail project and have been slightly amended for this project.

The BLUE trigger level is designed to provide an early warning to provide adequate time to assess and respond and is not linked to NSW rail safety standards. The RMG can review the adequacy of the BLUE trigger level during mining and adjust as agreed, without updating this management plan.

Trigger levels have been recommended by John Matheson & Associates (2008) and adopted in this management plan. The triggers refer to the following monitoring results:

- Opening of the bridge abutments
- Cracking of the masonry abutments and corbel
- Shear distortion of the bearing (if pre-greased joint is ineffective)
- Tilt of the abutment walls

Table 5.2 Risk Control Procedures for Thirlmere Way Overbridge

GENERAL TRIGGER LEVELS

Trigger Level	Description
GREEN	Observations within predictions. Operate as normal.
BLUE	Any of the following observations: 1. Any new cracking anywhere in the abutment or corbel 2. Horizontal abutment opening of 10 mm 3. Tilt of abutment walls reaches 3 mm/m 4. Damage to road pavement
YELLOW	Any of the following observations: 1. Horizontal abutment opening of 20 mm 2. Horizontal bearing shear distortion of 5 mm without horizontal slip movement 3. Tilt of abutment walls reaches 6 mm/m
RED	Any of the following observations: 1. 130 mm horizontal abutment opening movement of the bridge deck 2. Both abutment walls tilt outwards at 10 mm/m 3. Observations exceed any additional triggers recommended by ERG following reassessment in light of actual observations at time of Blue and Yellow trigger.

RISK ISSUE	TRIGGER	CONTROL PROCEDURES	TIMING & FREQ	BY WHOM?
General Procedures				
	GREEN	2D and 3D ground monitoring along rail corridor	Same frequency as per Main Southern Railway Subsidence Management Plan, or as agreed by RMG	Meadows
		2D and 3D monitoring Thirlmere Way (Remembrance Dr to Pitt Street)	Prior to commencement of each longwall Weekly once LW within 200 m of bridge (for LW25: after 700m of extraction) End of each LW	L&H
		2D structure surveys of abutment and wingwalls	Prior to commencement of each longwall Weekly once LW within 200 m of bridge (for LW25: after 900m of extraction) End of each LW	Meadows
		Measure gap across bridge deck above expansion joint	Prior to commencement of each longwall Weekly once LW within 200 m of bridge (for LW25: after 900m of extraction) End of each LW	Sunrise
		Track inspection by qualified track certifier	Same frequency as per Main Southern Railway Subsidence Management Plan, or as agreed by RMG	RMC
		Detailed building inspection by qualified building inspector	Weekly once LW within 200 m of bridge (for LW25: after 900m of extraction)	Sunrise
		Analyse and report results to RMG	Weekly	MSEC
		RMG assess monitoring results and consider whether any additional actions are required	Weekly once LW within 200 m until agreed to reduce	RMG

RISK ISSUE	TRIGGER	CONTROL PROCEDURES	TIMING & FREQ	BY WHOM?
Structural damage to Overbridge	**GREEN**	Follow general procedures (including mitigation works)	-	-
	BLUE	Notify ERG	Track Certifier within 15 minutes Rest of ERG within 30 minutes	RMC
		Inspect Overbridge	Immediately	RMC
		Conduct structural inspection and assessment of Overbridge	Within 24 hours	JMA
		ERG meet via teleconference and consider whether any actions are immediately required, which may include: - increase frequency of monitoring and inspections - commence regular survey of pre-greased bearing - install horizontal bar reinforcement to corbel - develop traffic management plan in consultation with Wollondilly Council in event that limitations might be placed on road traffic - develop additional triggers for Yellow and Red in light of actual observations	Following structural inspection	ERG
		Repair any new cracking to Bridge	As required	Tahmoor Colliery
		Report alarm and ERG decisions to RMG (incl ARTC, Tahmoor Colliery, DPI and MSB) and Wollondilly Council	Within 24 hours	RMC
		Report details of alarm and actions undertaken	Within one week	JMA
	YELLOW	Notify ERG	Track Certifier within 15 minutes Rest of ERG within 30 minutes	RMC
		Inspect Overbridge	Immediately	RMC & JMA
		ERG meet via teleconference and consider whether any actions are immediately required, which may include: - increase frequency of monitoring and inspections - install horizontal bar reinforcement to corbel (if not done so already) - limit road traffic in accordance with agreed traffic management plan in consultation with Wollondilly Council - develop additional triggers for Red in light of actual observations	Track Certifier within 15 minutes Rest of ERG within 30 minutes	ERG
		Report alarm and ERG decisions to RMG (incl ARTC, Tahmoor Colliery, DPI and MSB) and Wollondilly Council	Within 24 hours	RMC
		Report details of alarm and actions undertaken	Within one week	JMA
	RED	Notify ERG	Track Certifier within 15 minutes Rest of ERG within 30 minutes	Automated Monitoring System or Taylor Rail
		Stop trains and implement mandatory BOC responses as required	Immediately	RMC
		Stop road traffic over Thirlmere Way in accordance with traffic management plan	Immediately	Tahmoor Colliery
		Inspect Overbridge	Immediately	RMC & JMA
		ERG meet via teleconference and consider actions required to restart trains and road traffic	Within 15 minutes	ERG
		Report alarm and ERG decisions to RMG (incl ARTC, Tahmoor Colliery, DPI and MSB) and Wollondilly Council	Within 24 hours	RMC
		Report details of alarm and actions undertaken	Within one week	JMA
Damage to Road Pavement	**GREEN**	Follow general procedures (including visual inspection)	-	-
	BLUE (Cracking to road pavement)	Notify RMG (incl ARTC, Tahmoor Colliery, DPI and MSB) and Wollondilly Council	Within 24 hours	Tahmoor Colliery
		Repair pavement	As required	MSB
Encroachment of Overbridge within kinematic envelope	**GREEN**	Follow general procedures (including track inspection)	-	-
		Survey clearances to kinematic envelope	Start and end of each LW	Meadows

CHAPTER 6. REPORTING AND COMMUNICATION PLAN

Please refer to Main Southern Railway Subsidence Management Plan (GHD, 2008) for details.

CHAPTER 7. AUDITING AND REVIEW

Please refer to Main Southern Railway Subsidence Management Plan (GHD, 2008) for details.

CHAPTER 8. RECORD KEEPING

Please refer to Main Southern Railway Subsidence Management Plan (GHD, 2008) for details.

CHAPTER 9. CONTACT LIST

Organisation	Contact (*RMG Member)	Phone	Email / Mail	Fax
ARTC (Team Manager Moss Vale)	Ross Barber*	(02) 4868 0620 0419 466 143	rbarber@artc.com.au	(02) 4868 0637
ARTC (Work Group Leader Moss Vale)	Stuart Manray	(02) 4868 0615 0409 363 932	smanray@artc.com.au	(02) 4868 0637
ARTC (External Parties Officer)	Matthew Tyrell	(02) 6939 5432 0427 491 111	mtyrell@artc.com.au	(02) 6939 5437
ARTC (Track Patrolmen – Picton)	Darren Sharp	0419 690 624	N/A	(02) 4868 0637
ARTC (Signal Electrician – Moss Vale)	Various	0418 671 413	N/A	(02) 4868 0637
ARTC (Moss Vale)	Moss Vale	(02) 4868 0620	N/A	(02) 4868 0637
ARTC (Network Control)	Network Control in Junee	(02) 6924 9808	N/A	(02) 6930 5254
David Christie Geotechnical and Civil	David Christie*	0429 642 847	david.christie@bigpond.com	(02) 4758 7128
Department Primary Industries (Mineral Resources Division)	Gang Li*	(02) 4931 6644 0409 227 986	gang.li@dpi.nsw.gov.au	(02) 4931 6790
GHD (Principal Project Manager)	Graeme Robinson*	(02) 4979 9969 0410 455 911	graeme.robinson@ghd.com.au	(02) 4979 9988
John Matheson & Associates	John Matheson*	(02) 9979 6618 0418 238 777	jma.eng@bigpond.net.au	(02) 9999 0121
Meadows Consulting	John Rolles	0411 234 515	jrolles@meadowsconsulting.com.au	-
Mine Subsidence Board	Darren Bullock*	(02) 4577 1967 0425 275 567	d.bullock@minesub.nsw.gov.au	(02) 4677 2040
Mine Subsidence Engineering Consultants (MSEC)	Daryl Kay*	(02) 9413 3777 0416 191 304	daryl@minesubsidence.com	(02) 9413 3822
Sunrise Building and Property Services	John Schwarz	0400 390 058	sunbuilding@westnet.com.au	-
Rail Maintenance Contractor (RMC)	Mark Wroblewski* (Taylor Railtrack)	(02) 4272 1586 0408 614 622	mark@taylorail.com.au	(02) 42721576

Organisation	Contact (*RMG Member)	Phone	Email / Mail	Fax
Rail Maintenance Contractor (RMC)	Track Certifier Mobile Phone			
Rail Maintenance Contractor (RMC)	Tim Horan Site Supervisor			
Xstrata Coal Tahmoor Colliery Control Room	Control Room	(02) 4640 0111 (02) 4640 0176 0418 671 460	N/A	(02) 4640 0185
Xstrata Coal Tahmoor Colliery – Environment and Community Manager	Ian Sheppard*	(02) 4640 0100 0408 444 257	isheppard@xstratacoal.com.au	(02) 4640 0140
Xstrata Coal Tahmoor Colliery – Community and SMP Coordinator	David Clarkson*	(02) 4640 0133 0428 114 614	dclarkson@ xstratacoal.com.au	(02) 4640 0140

Appendix– Supporting Documentation

Please find enclosed the following supporting documentation:

JMA (2008). *Thirlmere Way Overbridge Bearing Triggers, Tahmoor*. John Matheson and Associates, Revision C, 2008.

Thirlmere Way Railway Overbridge Bearing Triggers, Tahmoor.

Prepared by John Matheson

Date: 20th October 2008

Revision C.

John Matheson & Associates Pty Ltd

Consulting Civil & Structural Engineers

2/1767 Pittwater Road

Mona Vale NSW 2103

Tel: 9979 6618

Fax: 9999 0121

Email: jma.eng@bigpond.net.au

Introduction

This report has been prepared at the request of Tahmoor Colliery. The purpose of this report is to consider the impacts of possible increased horizontal ground movements between the abutments and to determine trigger points for the bearings and masonry abutments in line with the other triggers being established for other infrastructure in the Tahmoor area that are affected by the impacts of mine subsidence. This report is based upon additional calculations carried out to assess the impact of possible additional horizontal movements upon the bridge bearings and corbel support.

The design of the recently constructed Thirlmere Way Railway Overbridge provided longitudinal bridge deck restraint by tying the bridge deck to the eastern abutment and allowing the western abutment move inwards/outwards from/to the eastern abutment in response to the original ground movements predicted. The bearing on the western abutment was constructed upon a pregreased slip joint to minimize the likelihood of generating tension forces in the bridge deck and additional horizontal forces at the top of each abutment, which would have added considerably to the anchoring force required and generated horizontal tensile stresses in the brickwork within the corbel. There is a low probability that the pregreased slip joint may be partially or completely ineffective and consideration has been given to this possibility in preparing the triggers for the bridge abutments and bearings.

The Australian Standard for bridge design AS5100: 2004 requires various load factors to be applied to dead, superimposed dead and live loads. However, to simplify matters, the average ultimate load factor for dead, superimposed dead and live loads are in the order of 1.75. The calculated excess capacity of the corbel is expressed as the ratio of the strain at brickwork rupture divided by the calculated strain due to bridge deck reactions calculated at ultimate loads and movements as defined in the report.

Calculated Excess Capacity = Rupture Strain Limit/Calculated Ultimate strain.

The proposed movement trigger levels are then established to achieve graduated levels of calculated excess capacity of the brick corbel with the red trigger being set at an intensity of bridge deck reaction and corresponding eccentricity at a level where the calculated tensile strain in the brickwork equals the strain limit of 0.05%strain.

The original trigger for bridge closure was based upon an unlikely horizontal movement of 130mm, which resulted in the elastomeric bearing moving toward the front face of the brick corbel establishing a point beyond which the bearing would become unsupported if further movement were to occur. Superimposing the original trigger with the triggers recommended in this report, it would be expected that severe live load restrictions would be placed upon the bridge if a movement of 130mm were to occur and the corbel were uncracked.

Calculations

The Australian Standard AS5100.4 Bridge Design – Bearing and Deck Joints is the design standard for the design of elastomeric bearings supporting the bridge deck and AS3700: Masonry is the standard for the design of masonry structures.

The design of the brick corbel for bearing and restraint forces must consider how these loads are applied through the bearing and how increased movements are likely to affect the corbel. The eccentric location of the bearing and its proximity to the front face of the corbel suggest that opening between the abutment walls is likely to be the controlling design consideration. The bursting and spalling stresses beneath the abutment bearing have been calculated using the methods normally used in the design of pre-stressed concrete end zones as the formulations provide a good representation of load dispersion and the tensile forces that are generated in response.

Based upon the original design and assuming a small abutment opening, the calculated peak bursting tensile stress of 0.37mPA located beneath the bearing (corresponding to a Type A crack) corresponds to a masonry strain of 0.0125% strain and the peak spalling stress, located behind the bearing toward the centre of the 960mm thick wall (corresponding to a Type B crack), is calculated to be 0.74mPa corresponding to a masonry strain of 0.025% strain at ultimate loads. Burland & Wroth report the rupture strain of masonry to be in the order of 0.05% strain and from this an excess capacity of 2.0 against corbel cracking due to spalling stresses at ultimate loads is calculated. The potential crack types A and B are shown in figures 2 & 3 in Appendix A of this report in addition to a corbel reinforcement detail in figure 4.

MSEC advise that opening movements between the abutments may develop during the mining of Longwall 25, though no significant opening movements were observed during the mining of Longwall 24B. Movement parameters have been established by iterative calculations for upper bound slip joint movement and bearing shear deformation. The calculated spalling strain due to a 30mm horizontal slip movement was found to be 0.0325% strain and the additional tensile strain in the brickwork due to 10mm bearing shear distortion was found to be in the order of 0.023%strain and superimposing these two strains results in a total tensile strain of 0.055%. Assuming a linear interaction between bearing slip and shear distortion, both of which can be measured on site, upper limits can be established that limit the calculated strains to 0.05% beyond which strain the brickwork is expected to crack, which thus establishes the red trigger.

In the event that the bridge abutments significantly open and the pre-greased slip joint proves to be partially or completely ineffective, it is possible to strengthen the corbel as per the corbel reinforcement detail in figure 4. Since the bridge deck is tied to the concrete capping beam on the eastern abutment the impacts of abutment opening and closing are expected at the bearing on the western abutment and therefore corbel strengthening should only be necessary at the western abutment.

Abutment closure movements are considered to be less significant since the centre of bearing reaction moves away from the front face of the corbel and restraint forces generated by bearing shear deformation are expected to cause perpendicular compression across any potential spalling Type B tension crack thereby reducing the tensile stress at the potential crack.

The masonry strain trigger levels adopted in this report have been established as follows in Table 1.

Trigger	Calculated Excess Capacity at Ultimate Load Levels	Comment
	2.0	Original Design with small slip movements
	1.75	
	1.5	
	1.0	Rupture of Modulus of Brickwork may be exceeded due to spalling tension

Table 1

Conclusions and Recommendations

The movements that correspond to the triggers specified in Table 2 are shown in Appendix A. These movements consist of horizontal sliding in the direction of the bridge span across the pregreased slip joint and/or horizontal shear deformation of the elastomeric bearing where the slip joint may be partially effective due to opening of the bridge span between the face of the opposing abutments caused by subsidence.

The movement triggers have been set to limit the calculated masonry tensile stress to the calculated excess capacity levels specified in Table 1 of this report. The movement triggers are recommended as follows:

Trigger	Trigger Movement	Comment
(green)	Original design, small movements predicted.	Monitor weekly during active subsidence period.
(blue)	Any new cracking anywhere in the abutment or corbel Horizontal abutment opening movement 10mm. Tilt reaches 3mm/m.	Monitor Weekly during active subsidence period. Immediate structural inspection and assessment. Inspect the bearing and monitor for deformation. Monitor rate of crack or tilt growth and consider limiting traffic loads. Consider installing additional horizontal reinforcing bars into the corbels.
(yellow)	Horizontal abutment slip opening movement 20mm without shear distortion. Horizontal Bearing Shear Distortion 5mm without horizontal abutment slip opening movement. Linear interpolation may be applied to the limiting trigger movements. Tilt reaches 6mm/m.	Monitor Daily during active subsidence period. Inspections to be carried out of the immediate bearing and brick corbel area including the main abutment wall. Monitor rate of tilt growth and consider limiting traffic loads.
(red)	130mm horizontal abutment opening movement of the bridge deck. Both abutments tilt outwards at 10mm/m.	Stop Trains and road traffic

Table 2

It is important to note that the calculated excess capacities of the brickwork are based upon an assumption that the original structure was appropriately constructed and the there are no unusual, areas of weak mortar or unexpected residual stresses within the brickwork that may initiate earlier cracking than has been predicted herein. In essence, there is an expectation that load limits may need to be set for vehicle traffic using this bridge if a blue or yellow trigger is reached. The calculated loads have been based upon an S/M1600 truckload and consideration may be given to limiting vehicles to a specified number and intensity of axle loads in such circumstances to maintain a minimum level of service to the community.

The effects of abutment closure upon the movement of the bearing over the slip joint and/or bearing shear distortion were noted earlier in the report to be less critical than the effects due to abutment opening movements. Abutment closure movements are expected to result in the movement of the bearing away from the face of the corbel and a possible compensatory compressive stress perpendicular to a possible Type B crack reducing the calculated spalling stress rather than adding to the spalling stress as is predicted to occur if abutment opening occurs. Should the abutments close by more than 100 mm, it is expected that the steel bridge deck will bear against the concrete capping beam and generate an outwards thrust against the capping beam. This situation was anticipated in the original design and the abutment wall was vertically reinforced for this scenario. Some horizontal cracking of the reinforced abutment wall may occur, which can be monitored and is not significant. No triggers are recommended for abutment closure.

The beneficial effects of the existing brickwork reinforcement have not been considered in preparing this report since the reinforcement was installed for the purpose of tying the concrete capping beam to the brickwork rather than for specifically tying across the possible Type A and B tension cracks. This existing reinforcement is expected to raise the calculated excess capacity of the brick corbel and should offer additional ductility should a crack initiate. However, the exact termination points of the existing brick reinforcement are not precisely known (they were drilled essentially to the length shown on the drawings but there may be some variation in cut-off position). It is therefore recommended that, in the event of significant abutment opening, a row of galvanised N16 bars at 300mm centres be drilled from the track side of the western abutment wall and epoxied into the corbel to maintain 40mm cover as shown in figure 4 in Appendix A to reinforce across a potential Type B crack.

An appropriate timing for this work to be carried out is when the blue or yellow trigger is reached, depending on the rate of movement and the amount of additional movement that is expected to occur. This is expected to allow for sufficient time to mobilise resources to drill and epoxy the additional horizontal reinforcement into the wall before the tensile strain in the wall exceeds the limit of 0.05% strain. Given that only the western abutment would require reinforcement, the works could be carried out under management with one track being kept open during the works period.

It is reminded that the recommendations in this report are made in the unlikely event that the designed pre-greased slip joint proves to be partially or completely ineffective. If the slip joint performs as designed, the original 130 mm abutment-opening trigger applies.

Appendix A

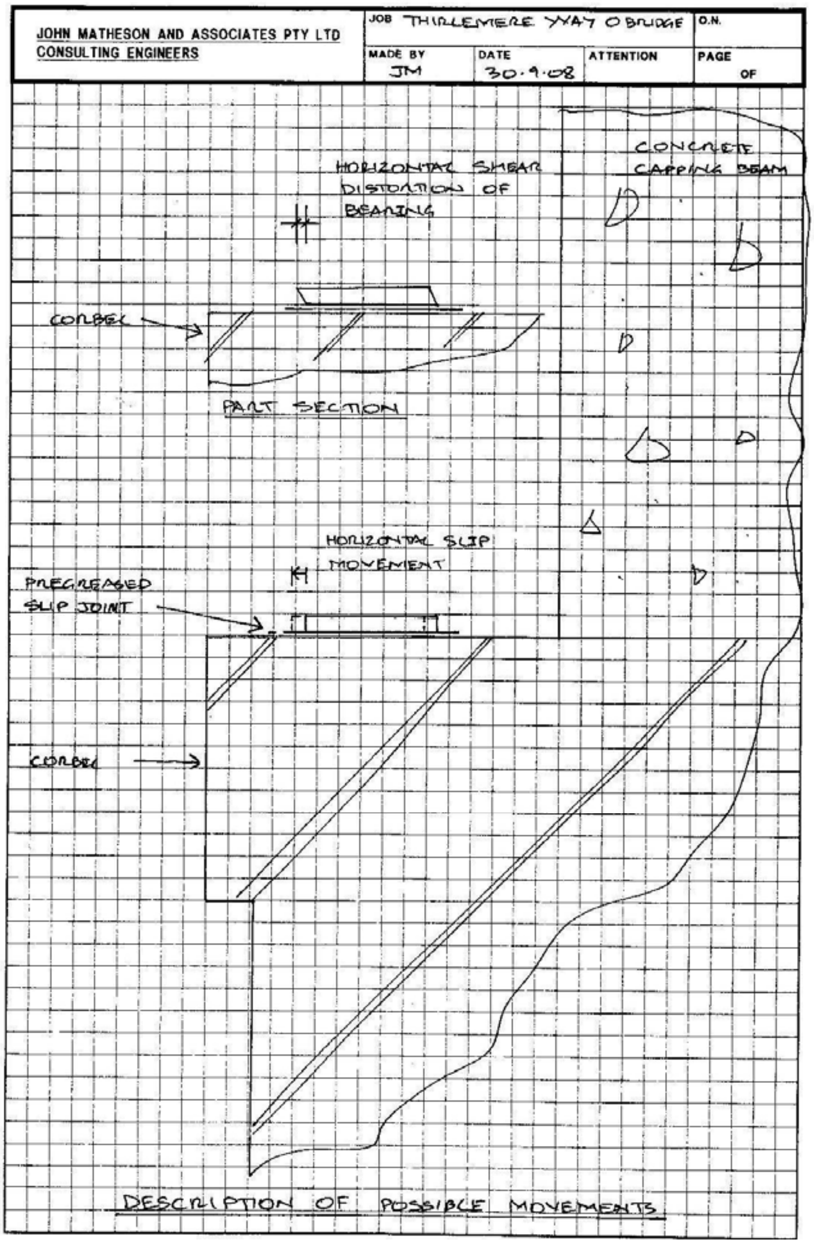

Figure 1

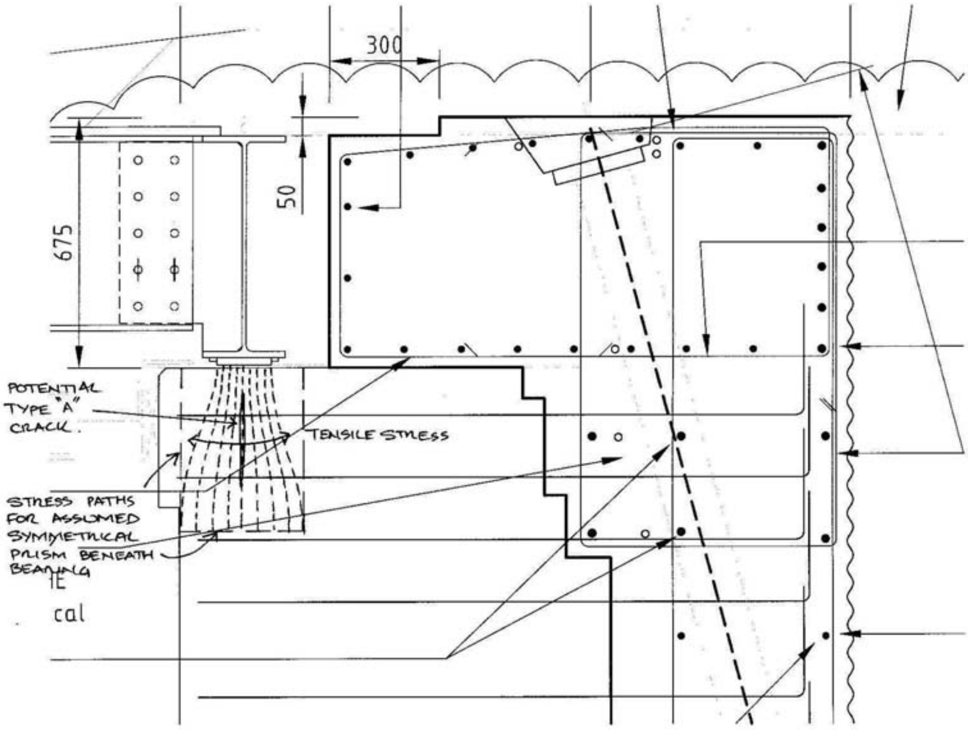
300
50
675
POTENTIAL TYPE "A" CRACK.
TENSILE STRESS
STRESS PATHS FOR ASSUMED SYMMETRICAL PRISM BENEATH BEARING
cal

Figure 2

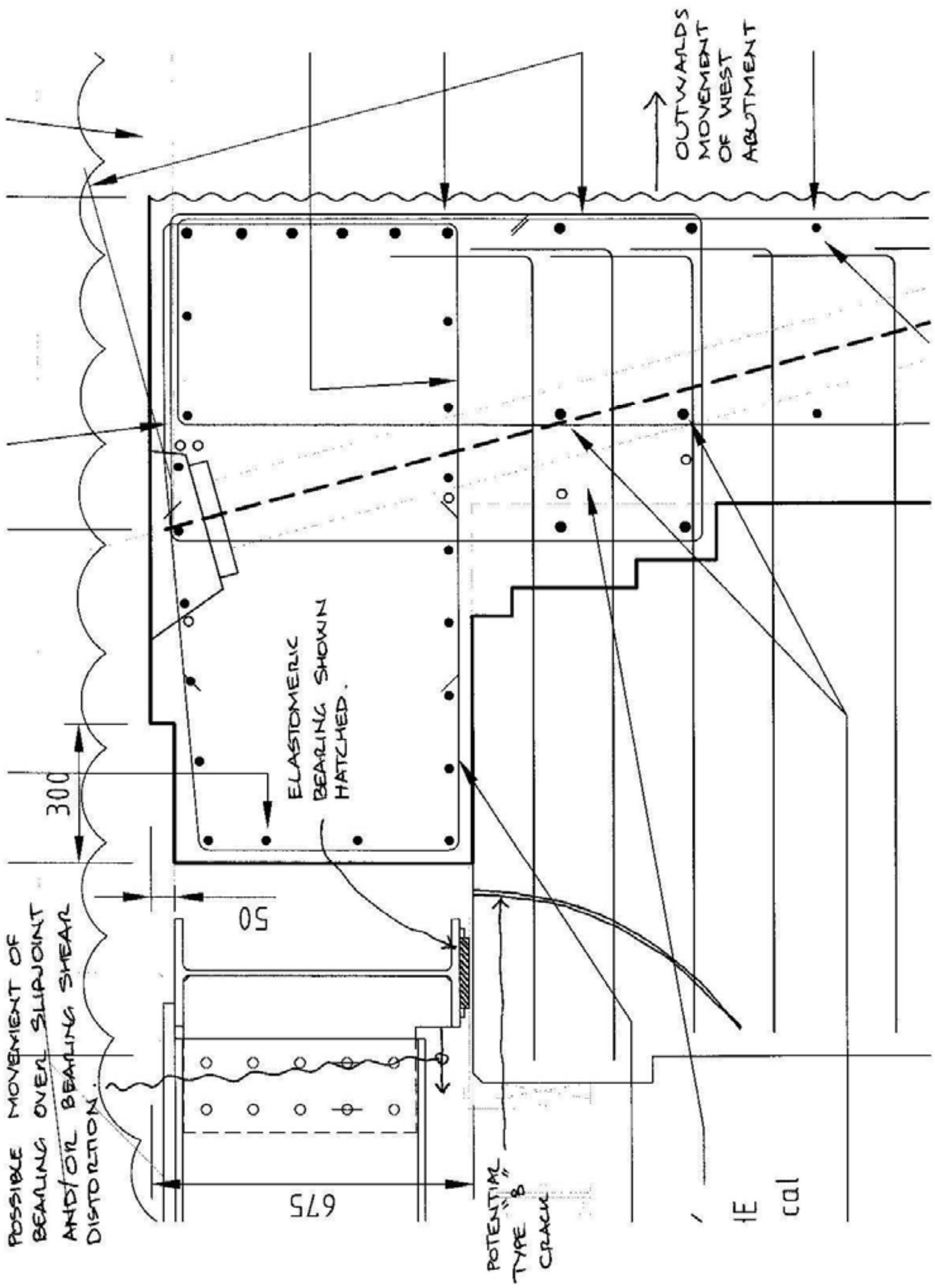
OUTWARDS MOVEMENT OF WEST ABUTMENT
ELASTOMERIC BEARING SHOWN HATCHED.
300
50
675
POSSIBLE MOVEMENT OF BEARING OVER SLIPJOINT AND/OR BEARING SHEAR DISTORTION.
POTENTIAL TYPE "B" CRACK

Figure 3

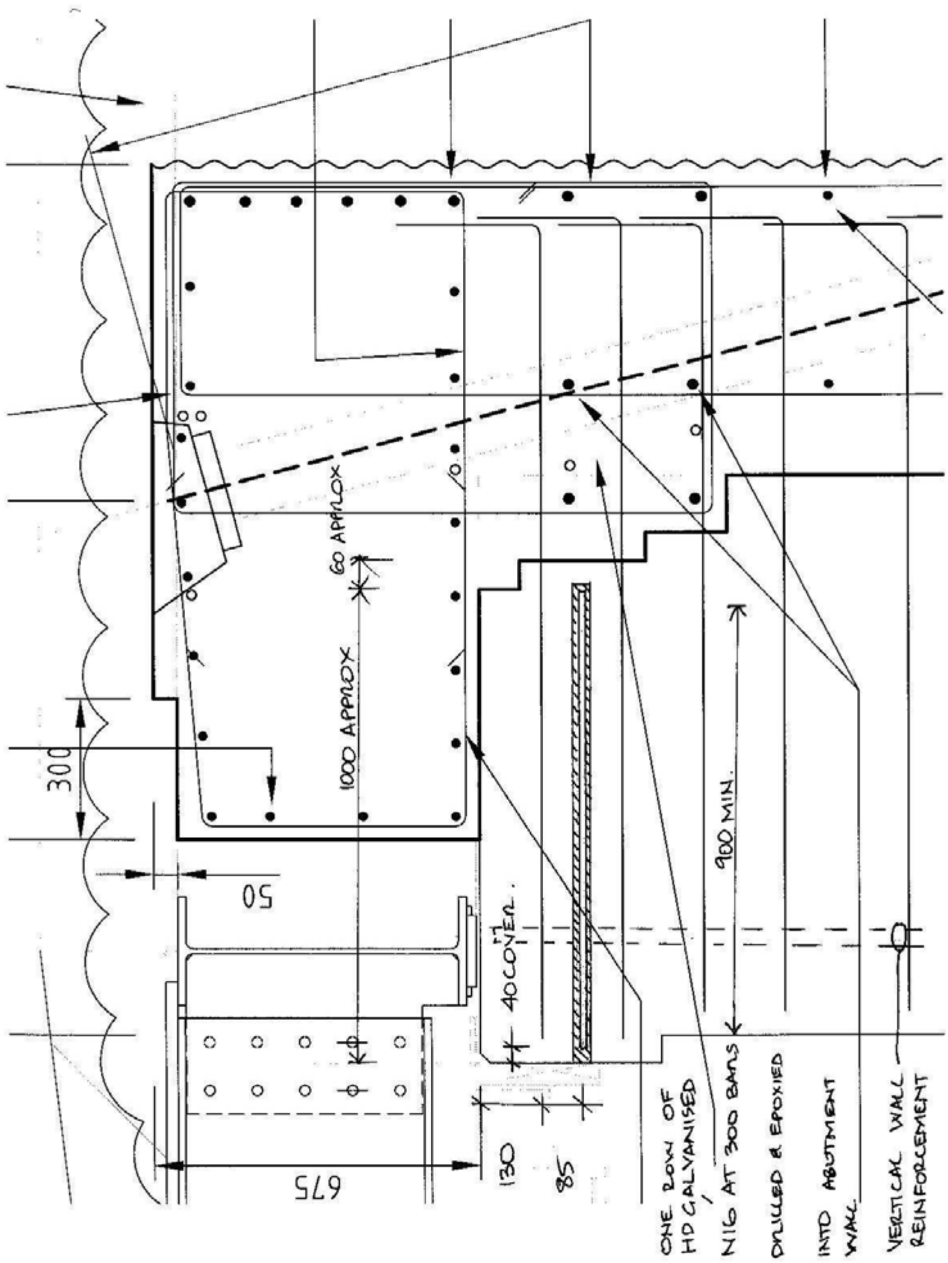
300
50
675
130
85
1000 APPROX
60 APPROX
40 COVER
900 MIN.
ONE ROW OF HD GALVANISED
N16 AT 300 BARS
DRILLED & EPOXIED
INTO ABUTMENT WALL
VERTICAL WALL REINFORCEMENT

Figure 4